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REMOLDING OF SOILS UNDER AIRCRAFT LOADING - LITERATURE SURVEY AND RESEARCH PLAN

PHASE II

by

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The behavior of unsurfaced runways under aircraft movement has been studied for a number of decades. A good number of the design nomograms developed have been intended for the prediction of runway performance in a variety of soils and under diverse loading conditions. At the same time, a comparison of the design values with the actual behavior of the runways in the field, indicates a very large scatter of the results.

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REMOLDING OF SOILS UNDER AIRCRAFT LOADING - LITERATURE SURVEY AND RESEARCH PLAN

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Chapter 1: Introduction

1.1 General

The behavior of unsurfaced runways under aircraft movement has been studied for a number of decades. A good number of the design nomograms developed, have been intended for the prediction of runway performance in a variety of soils and under diverse loading conditions. At the same time, a comparison of the design values with the actual behavior of the runways in the field, indicates a very large scatter of the results.

One of the main reasons for the great uncertainty associated with the prediction process, can be found in the changes in soil strength which take place following aircraft movement over the runway. The soil shear strength is one of the more important input parameters in the process of predicting the performance of an unsurfaced runway under aircraft movement. The accepted prediction methods carry out strength tests by means of various field instruments (DCP, Airfield Cone Penetrometer, and others) before the use of the soil strip begins. The strength values thus obtained are then used as an input in the prediction process.

Under certain conditions, one can expect changes in the soil strength, as a consequence of accumulative aircraft coverages. This change in soil strength leads to a consequent change in the soil's ability to bear passing traffic.

The literature which deals with soil mobility presents a field method for evaluating the change in soil strength during movement. The use of this method may yield a corrected soil strength value which can be used as input instead of the initial soil strength. This method however suffers from considerable limitations and is confined to cohesive soils only.

The literature does not include any practical field method for predicting the expected strength changes in granular soils.

The main objectives of this phase of the research (Phase II, see Fig. 1.1) are:

a. A laboratory investigation of the soil strength change potential of various soils during shearing.

b. Identification of change processes in the strength of various soils (with emphasis on granular soils) during repeated wheel movements. An attempt will be made to evaluate the dimensions of the soil section undergoing remolding and the rate of strength change during repeated passes of the wheel.

c. An attempt will be made to establish a field method intended to evaluate the extent of the expected change in the strength of various soils under the movement of aircraft wheels.

The success of this stage of the research will allow improvements to be made in the existing design processes, so as to obtain more reliable predictions of the performance of unsurfaced runways under aircraft movement.

The present report is the first in Phase II of the research and includes the following main subjects:

a. A survey of the existing literature dealing with remolding phenomena in soils, the influence of aircraft wheel movement on soil strength and the plastic deformations accumulated during cyclic loading of soils.

b. A presentation of the general principles of the proposed model for performance of remoldable soils.

c. Guidelines for the planned laboratory program including a preliminary field method for evaluating the remolding potential of soils.

1.2 Structural Distrurbance and Strength Change

The term "remolding" refers to changes in the internal structure of the soils as compared to its initial, original state. External intervention leads to changes in the relative positioning of the soil particles, the orientation of the particles and other factors which lead to changes in mechanical and other qualities of the soil.

Various sources in the literature have addressed the issue of the change of soils strength during shear. A great deal of research has been done on clayey soils which, under certain conditions, display a peak strength in small deformations followed by a sometimes dramatic decrease in strength, to the point where, under large deformations, the strength attains a value termed "residual strength".

In other soils, and especially in dense granular soils, the shear process is accompanied by dilatation phenomena, i.e., an increase in volume in the sheared area. The dilatation phenomenon too is a form of disturbance of the soil structure which effects the soils structural strength as the shearing continues.
Loose soils, on the other hand, may become denser and stronger following the shearing action of the wheel load. The movement of aircraft wheels over an unsurfaced runway creates shear activities in the soil beneath the wheel and its immediate surroundings. The plastic shear causes a relative movement among the soil particles and the disruption of the soil's initial structure.

The changes in the soil strength lead to changes in the soil's ability to withstand the wheel movement and thus also in the number of coverages until the formation of ruts in accordance with the failure criteria. The identification of the remolding potential of various soils under aircraft wheel movement and the rate of the expected change in strength under various conditions, are therefore very important issues in the prediction of unsurfaced runways performance.

1.3 Structure of the Report

This report is the first report in the context of phase II of the present research, and mainly includes a literature survey, as well as the main assumptions and guidelines for proceeding with the research, both experimentally, in the laboratory, and theoretically.

Chapters 2 and 3 of the report will include a literature review of soils' remolding phenomena and related issues.
Chapter 2 will itemize the various factors which lead to different types of the soil remolding phenomena. The rate of change in soil strength during shear and the behavior of the various strength functions vis a vis the deformation, depend to a great extent on the type of soil, the structural bonds and the relative positioning of the soil particles. An attempt to evaluate the potential for change in soil strength during remolding requires a preliminary understanding of the various factors which effect the process.

Chapter 3 will discuss the particular effects of cyclic and wheel loads exerted by the moving wheel on the unsurfaced runway and their effect on strain and remolding phenomena in soils. This chapter will also include a brief description of observations made during landing exercises in Israel, and will survey the approach of present design methods to the remolding phenomena in soils under repeated wheel loads.

Chapter 4 will summarize the main points of the literature review. The general outlines of a proposed model for the behavior of remoldable soils under aircraft wheel landings, will be depicted, including a preliminary frame for a field method intended to identify remolding potential in soils.

A research program was built on the basis of the proposed qualitative model. The program includes theoretical and empirical stages meant to investigate remoldable soil properties, in order to quantify and to prove the validity of the model under laboratory and field testing.

The main stages of the research program will be presented in Chapter 5, which will also include a summary of the present report.

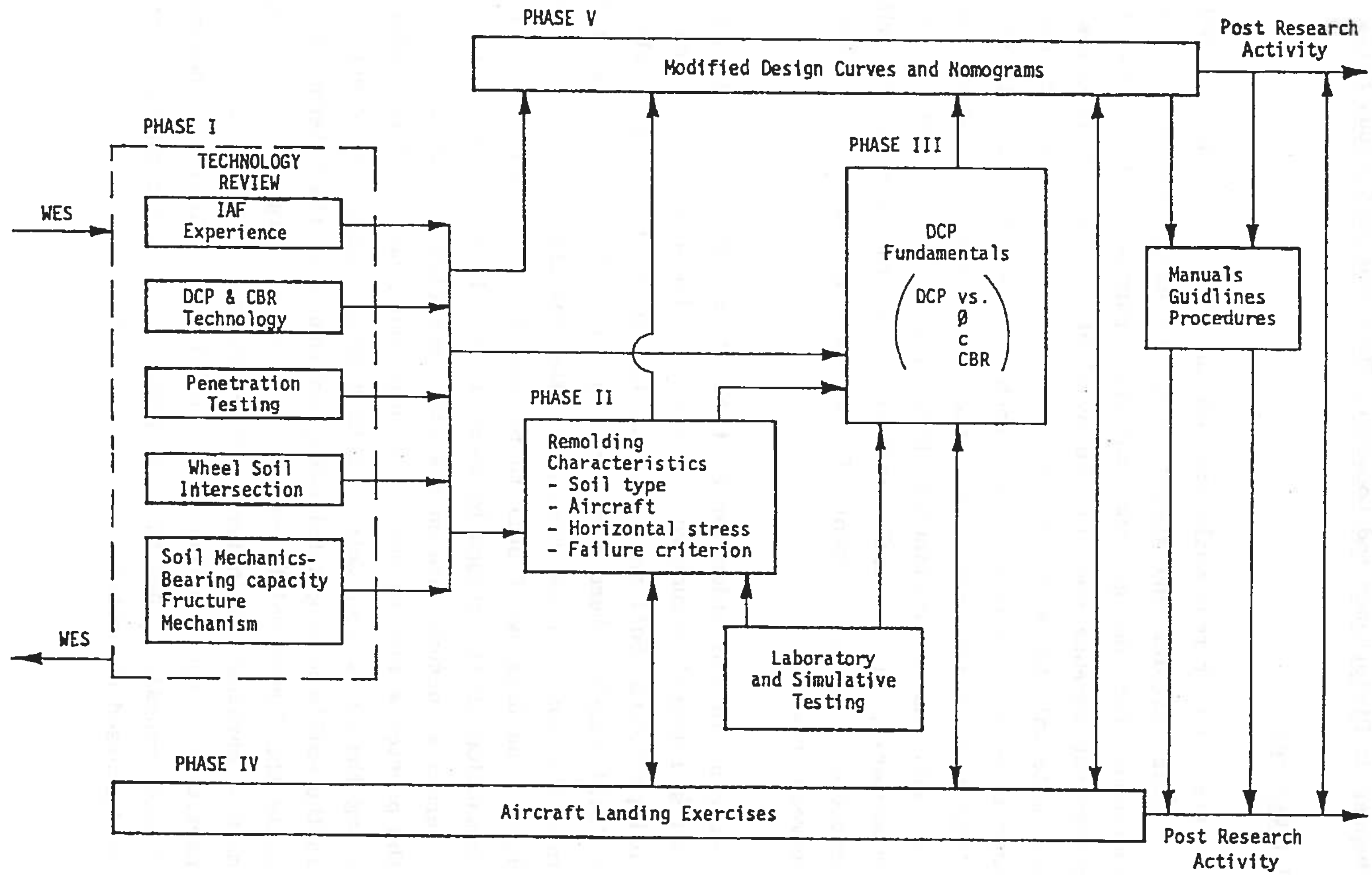


Figure 1.1 Summary of research methodology. (Ref.7).

Chapter 2: Disturbance and Loosening Phenomena in Various Soils

2.1 General

Despite the fact that soils are made up of a combination of individual particles, liquids and gasses, they are almost always treated as a continuum for design and analysis purposes. This approach is convenient because most of the available mechanical theories regard the material as a continuum, and because in most of the cases associated with soil-structure problems the scale is big enough to allow such an approach. Nevertheless, the properties which represent the soil as a continuous body (such as the soil's strength parameters), are directly influenced by the nature of individual particles, the arrangement of the particles and the forces acting between them.

Remolding in the wider sense of the term refers to a change in the soil's internal structure and the relative position of the various soil particles. Soil remolding may take place during the collection of a field sample, where the very insertion of the sampling instrument into the soil causes changes in the positioning of the particles, during the digging of soil to be used for refilling, during kneading compaction of the ground by means of a roller, and more. The initial internal structure between the soil particles, the basic qualities of the particles such as their size and shape, and the forces which apply among the particles, determine to a great measure the exent of change in the soil's strength following the change in the internal structure. While the forces which are active among the larger soil particles are mainly physical in nature, the smaller particles, such as clayey particles, are subject to additional forces; chemical and chemical-physical. Due to this, granular soils and cohesive soils will be discussed separately.

2.2 Granular Soils

A number of factors may lead to changes in the strength parameters of a granular soil while disturbing its initial structure:

a. Breakage of the inter-granular bond and changes in density

When a granular soil undergoes shear under drained conditions, the shear process is often accompanied by a change in density. For a certain type of soil it is possible to observe a direct correlation between the confinement pressure exerted on the sample and the density (termed the critical density) which the sample will attain after a certain shear deformation. Fig. 2.1 (Ref. 1) displays shear stress vs. deformation and shear strain vs. void ratio curves, for two samples of the same type of sand, wherein one sample was in a dense state while the other was in a loose state, at the beginning of the test.

The dense sample displayed the dilatation phenomenon while the loose sample densified and its void ratio decreased during shear. After a certain shear deformation both samples attained the same void ratio which is defined as the critical void ratio. Fig. 2.2 presents an example of the correlation between the confinement pressure and the volumetric changes obtained for a given type of sand. Each confinement pressure yields a specific critical density value and this is the density which the soil will attain regardless of its initial density.

As there is a direct correlation between the density of the granular soil and its strength parameters, the conclusion is that in the area in which the shear takes place under low confinement pressure the residual strength following the shear will be lower, and the obverse goes for an area sheared under a high confinement pressure.

In addition to the changes in density and strength which are expected during shear of granular soil, there is the added element of the breaking of the inter-granular bond.

Fig. 2.3 illustrates this issue where in order to create a shear plane in the granular soil (based on the assumption that the strength of the particles themselves is very high) the bond between the particles must be overcome. This situation leads to an increase in the soil's resistance to shear and to a peak phenomenon in the shear-stress deformation curve. Following a certain deformation, the strength decreases once more until it stabilizes at the residual strength appropriate for the confinement pressure under which the shear was conducted. The peak strength phenomenon becomes more salient the higher the soil density before the passage of the wheel is, relative to the soil density appropriate to the confinement pressure under which the shear is conducted. When the initial soil density is significantly lower than the critical soil density, the peak phenomena is not observed and the shear takes place in a strain-hardening mode. (See Fig. 2.1, for loose soil).

The effect of the change in density and the inter-granular bond on the behavior of the soil during shear can be summarized in the following way: The parameters which effect the strength of a given soil are the initial soil density, the confinement pressure to which the soil is subjected during shear and the extent of shear deformation to which the soil is subjected.

In the context of structural strength and density changes, it is important to mention the issue of the behavior of granular soils which are saturated or close to saturation during a fast shear. This is the situation which holds during the loading of an aircraft wheel. The fast loading makes it difficult for the water to drain from the soil during the passage of the wheel, and this may lead to a state of undrained shear in most types of granular soils, except perhaps for gravels and coarse sands which have high permeability values. In the dense granular soils which tend to undergo dilatation during shear, the saturated soil will develop negative pore pressures which increase the effective stresses and the soil's shear strength. As opposed to this, in loose soils which tend to compress during shear, a liquefaction process may occur where the external load is totally transferred to the pore waters and the structural strength of the aggregates disappears. It may be difficult to identify the danger of liquefaction in sandy soils as the risk of liquefaction only occurs under very high loading rates (wheel loadings). These loadings are hard to simulate during field testing, especially when the strengthening of the soil, once the load is removed, is very fast in these soils.

The issue of saturated granular soils will not be addressed in the context of the present research.

b. Cementation and Capillary Stresses

In addition to the strength which arises from the internal structure and friction forces among the aggregates in the granular soil, there are additional factors which may increase the initial strength of these soils.
Fig. 2.4 presents a magnified sketch of the particles in a granular soil, among which cementation forces are at work. Tiny amounts of cement materials, which are sometimes almost non-visible in the gradation tests, concentrate at the contact points between aggregates, usually during drying processes, leading to cementation of the aggregates. The cementation process increases the soil shear strength and the additional strength is essentially cohesive in nature.

A phenomenon which is similar to the cementation phenomenon is that of the accumulation of negative pore pressures between the aggregates in the soil. Negative pore pressures are formed as a consequence of a combination of the adhesion of water onto the surfaces of the aggregates and the water surface tension characteristic. During the drying process, membranes of water under negative pressure are formed at the contact points among the aggregates in the soil, (see Fig. 2.5) leading to an increase in the effective stresses acting in the soil and thus also in the soil shear strength. The additional strength can also be seen in the strength envelope curve as an addition to the cohesion. This cohesion is called "apparent cohesion" as there is no true bonding phenomenon between the particles, but only an increase in the effective stresses.

The phenomenon of capillary pressures is more significant in fine soils than in coarse ones. While in gravel and in coarse sands this phenomenon has no significant effect on the soil strength, in fine sands and silts a significant increase in strength can sometimes be discerned as a consequence of capillary pressure. There is a direct relationship between the additional strength resulting from capillary pressure and the specific surface of the aggregates. Both the capillary pressure phenomenon and the cementation phenomenon (in most actual cases) are very sensitive to the soil water content and they are mainly evident when the water content is relatively low.

Despite the fact that the cementation phenomenon and the capillary pressure phenomenon are usually distinguished one from the other, in many cases the strength of the cementing material concentrated between the coarse aggregates is the result of capillary pressure among the aggregate's fines.

In these cases there is an especially high sensitivity of the cementing element to changes in moisture as the addition of even an extremely small amount of water relative to the weight of the entire soil sample. This may have considerable influence on the fine material concentrated in the contact points. The addition of water cancels out the suction in the fine material and diminishes the cementing forces between the aggregates.

Fig. 2.6 presents a typical shear stress-deformation curve for a soil with a significant cementing element. The initial strength is relatively high while later on, after a small shear deformation, a kind of "breakage" occurs in the cemented aggregate structure and the strength decreases quickly down to the residual strength value.

The cementation phenomenon and the capillary pressure phenomenon, under low moisture content, both disappear after shear and do not reappear unless the process of wetting and drying is repeated. This phenomenon stems from the fact that the tiny amounts of cementation agents have been detached from the contact points between the aggregates.
The phenomenon of the high initial strength of granular soils when water content is low, is rather frequent in arid and semi-arid climates in sandy areas, deserts, etc. Reference to the initial soil strength alone without an inspection of the soil's remolding potential, may lead to a false evaluation of the soil's ability to carry aircraft and other traffic.

2.3 Cohesive Soils

The behavior of clay soils is far more complex than that of granular soils. In contrast to granular soils, where the inter-granular forces are mostly mechanical, clay soils combine mechanical forces with the forces of various chemical ties. The multiplicity of factors and forces which operate among the particles, leads different types of clays to behave in totally different ways, in accordance with their structure and the relative dominancy of the various forces.

Like other clay behavioral phenomena, soil remolding and behavior during shear are also complex phenomena which change from one type of clay to the next. A number of basic factors may contribute to strength change in cohesive soils during shear:

a. Inter-granualr strength - this phenomenon is very similar to that which occurs in granular soils, but on a much finer scale. Inter-granular bonding appears in over-consolidated cohesive soils in which the clay particles are locked in a dense structure. The shear process which breaks the interlocking between the clay particles causes local dilatation at the failure area and the formation of shear planes.

The behavior of the shear stress-deformation curve for the over consolidated soil will be similar to that of dense granular soil (see Fig. 2.1a) while the normally consolidated soil will behave more like loose granular soil.
The more over consolidated is the soil, the more pronounced is the tendecy to attain a high peak strength followed by a decrease resulting from "breakage" of the particles' dense structure.

When the soil is in a saturated state, negative pore pressures are generated during the shear process of the over-consolidated soil. These pressures increase the effective stresses in the soil, and the resulting peak strength of the soil.

b. Clay bonding strength - is a phenomenon exressed by a shearing behavior similar to that obtained in over-consolidated soil. The source of this phenomenon is in strong chemical ties between clay particles formed by the presence of various materials such as Carbonate, Sillica, Aluminum, Iron Oxides and various organic compounds. These bonds may cause the material to have a very high peak shear strength. Such a soil usually functions in a rigid way, wherein there is a kind of "breaking" of the structure once the peak strength point is passed.

A bonded clay, is a clay in which the internal structure among the clay particles, (or among groups of particles) is usually a floculated structure. The floculated structure, unlike the dispersed structure, (see Fig. 2.7) is a more "open" structure in which particles are bound to each other mostly by edge to edge or edge to surface bonds. (In the dispersed structure, the bonds between the particles are mostly surface-to-surface). The forces of attraction between the particles, which are created as a consequence of the nature of the clay particles and the presence of other materials, create a rigid structure whose strength is usually much higher than that of the dispersed structure. The shear behavior of a bonded clay will be expressed by a high initial strength resulting from the rigid structure, which later breaks so that the strength of the clay decreases significantly.

A phenomenon which is typical of structural cohesive soils is that of liquification. This phenomenon is characteristic of structured clay soils whose internal composition is very "open". The high void ratio permits high water content, while the bonds between the clay particles preserve the soil's strength. In extreme cases, (depending on the type and character of the clay, and on other factors as well) the shear process may lead to a collapse of the soil structure, to the transfer of stresses from the particles to pore pressure, and sometimes even to the transformation of clay particles into a suspended state within the pore water. In this respect, the ratio between the soil's peak strength and its residual strength is termed the soil's sensitivity, as described in the following equation:

$$S_t = \frac{t_f}{t_r} \qquad [2.1]$$

where,

t_f is the peak shear strength.

t_r is the residual shear strength in large deformations.

Table 2.1 (taken from Ref. 3), classifies clays according to their level of sensitivity. This table emphasizes the pronounced decrease of soil strength that may occur in certain types of soils.

Table 2.1: Classification of Clays According to Their Sensitivity Values

Level of Sensitivity	S_t
Insensitive	~1.0
Slightly sensitive clays	1-2
Medium sensitive clays	2-4
Very sensitive clays	4-8
Slightly quick clays	8-16
Medium quick clays	16-32
Very quick clays	32-64
Extra quick clays	>64

c. <u>The orientation of the clay particles</u>

The shape of the basic clay particle is long and flat. In the natural soil, the clay particles are arranged in different directions and in different structures in accordance with the forces acting among them. When shear deformations develop in the soil, those particles which are arranged in a direction opposing the direction of shear, produce forces which increase the shear resistance. During the shear, and as a result of it, the particles in the vicinity of the shear plane tend to gradually arrange themselves in a direction which parallels the direction of the shear.

Fig. 2.8a presents a shear stress-deformation curve for a clayey soil. After attaining the peak shear strength, strength gradually decerases under greater deformations until it reaches its residual value. This phenomenon which appears in both over-consolidated and normally consolidated clays, adds to the shear strength of most clay soils.

It is important to note the anisotropic nature of this phenomenon as compared to other phenomena related to changes in soil strength during shear. Stopping the shear of the soil once it has attained the residual strength value and proceeding with the shear in a direction perpendicular to the previous direction, will yield a high shear strength value. An additional shear deformation will be required in order to cause further turning of the clay particles in a direction parallel to the direction of the shear. It is important to note that the orientation of clay particles is relevant in soils where the clay component is dominant (over 40%). In soils with a low clay content (lower than 20%), this factor has no effect on the behavior of the soil. (See Fig. 2.8b)

2.4 The Rate of Change in the Soil Strength of Potentially Remoldable Soils.

In order to evaluate the behavior of the runway, it is necessary to know the soil strength which the aircraft wheel encounters at every stage of the runway's life cycle. Assuming that soil structural remolding phenomenon indeed causes a change in strength, it is necessary to find a way to evaluate the rate of soil strength decrease during aircraft movement over the soil. Chapter 3 will address the issue of the effect of the wheel's movement on the remolding. This section will present some of the experimental relations found which appear in the literature between the amount of the soil's deformation and its strength.

Fig. 2.8, taken from Ref. 10, clearly illustrates the behavior of the strength change phenomenon in clayey soils, divided according to its elements. The peak strength is attained at small deformations (tests were conducted by means of a reversal shear box and a ring shear instrument) in the range of 0.5-3 mm in over-consolidated soil, and 3-6 mm. in normally consolidated soil. The decrease in strength which stems from the breaking of the inter-granular bond, is fast and sharp and is expressed in low deformation values of 4-10 mm. Once the effect of the inter-granular bonding has been cancelled, the effect of the particles' orientation (in soils with high clay fraction) is felt and a slow and gradual decrease in strength can be observed down to the residual strength which is attained at a deformation range of 100-500 mm.

The distinction between the strength segment arising from the inter-granular bonding and that arising from particle orientation, is based on the comparison of the behavior of over-consolidated samples and the normally consolidated samples of the same clay, wherein the normally consolidated clay does not display the inter-granular bonding phenomenon.

Similar to the inter-granular bonding phenomenon, the phenomenon of bonding strength stemming from a special structure and the existance of cementing ties, is a "rigid" one as well, and the breaking of the structure takes place at very low deformations. Similar results were obtained with other natural clays as reported by Ref. 10 (see Fig. 2.9) and Ref. 11. Fig. 2.10 presents an example of the results of experiments which were conducted in a clayey soil, as reported by Ref. 10. For the undisturbed clay sample, one can clearly observe the "breaking" effect of the bonding phenomenon, wherein the peak is reached at deformations of less than 1mm. The post peak decrease in strength is very fast and ends at a deformation range of 6-8mm. Under the given test conditions, the soil strength in this clay decreased by about 50% within this deformation range.

Additional structural strength (stemming from cementation, chemical bonds and negative pore pressures) may be sensitive to confinement pressure conditions created during loading, probably because of the speical structure and the fragile bonds. Conditions of high confinement pressure (see Ref. 8) even with no shear stress, may cause a substantial decrease in strength due to the breaking of bonds and the collapse of the stable structure.

A number of researches examined the behavior of the shear stress - strain function in various granular soils which include cementation, either natural or artificially contrived. Ref. 22 investigated the behavior of 4 soil types with natural cementation as well as sandy soil with the addition of 2% and 4% Portland cement, under static loading in a triaxial shear device. Figures 2.11 a and b present the results of the tests for various confinement pressures in sands with strong and intermediate natural cementation, respectively. The soil with the high cementation displays brittle behavior, while the soil with the intermediate cementation displays relatively ductile behavior, when the tests are carried out under high confinement pressure.

Similar results have been obtained in other sand types. The less the cementation, the lower the confinement pressure under which the ductile behavior of the soil is displayed. The authors ascribe this behavior to the variable contribution of the frictional strength, depending on the confinement pressure. Under higher confinement pressure, the frictional part in the strength of the material becomes dominant and the effect of cementation becomes less salient, particularly in those cases where the cementation is not high. Another reason, not mentioned in the above reference, which may provide an explanation for the ductile behavior under high confinement pressures, is the "breaking" of the cementation as a consequence of the exertion of the confinement pressure itself. Similar results have been obtained by Ref. 22 in an investigation of sandy soil with Portland cement added. When the failure envelopes derived from these experiments are constructed (see Fig. 2.12), an internal friction angle in the range of 37°-49° is obtained, and the cohesion resulting from the cementation is in the range of 0.1 kg/cm^2 in low-cementation soils and up to about 3.5 kg/cm^2 for high-cementation soils (natural soil).

Ref. 29 has examined the strength and the carrying capacity of sandy soil with an addition of 1% Portland cement, with the aim of using the results to derive the behavior of soils with natural cementation. The added strength obtained as a consequence of the cementation was small and the behavior during shear was also "brittle" under low confinement pressures and relatively ductile under greater confinement pressures. The added cementation provided a cohesive factor of no more than about 0.25 kg/cm^2, but despite this fact the added cohesion greatly influenced the results of 5x30cm foundation model loading experiments, wherein the carrying capacity of the foundation increased almost 8-fold. The explanation for this phenomenon can be found in the relatively small dimensions of the foundation and the absence of soil pressure above the bottom of the foundation, factors which served to convert the cohesion factor in the total carrying capacity of the foundation to be the dominant factor. There is no doubt that this point is greatly significant in relation to the movement of the wheel over the soil. The experimental results indicate that there will be no change in the friction angle of the sand (about 34° in this case) as a consequence of the added cementation. An earlier research (Ref. 23) of naturally cemented soils of varying levels of cementation, has produced similar results.

Ref. 30 has made use of sand stabilized with varying quantities of Portland cement, with the aim of emulating the behavior of naturally stabilized soils. Fig. 2.13 illustrates the change in the value of the brittleness coefficient B_c, as a function of the confinement pressure during shear, the amounts of the cementing material and the relative density of the stabilized soil.
The brittleness coefficient B_c is defined as follows:

$$B_c = \frac{S_{peak}}{S_{resid}} \qquad [2.2]$$

where,

S_{peak}- Peak material strength.
S_{resid}- The material's residual strength.

From figure 2.13, it can bee seen that the lower the confinement pressure during shear and the higher the cement content, the greater the value of B_c.

Ref. 30 also analyzes the results of previous experiments in an attempt to evaluate the stress-strain behavior of soils with cementation. Figures 2.14 a and b illustrate the characteristic behavior under shear of strongly and weakly stabilized soils, respectively. Once again, the "brittle" behavior is more pronounced under low confinement pressures as compared with more ductile behavior under high confinement pressures. Elastic linear behavior, almost up to peak strength, was mainly observed in soils with high cementation. Peak strength is usually achieved at deformations of 0.3-0.8%, followed by the characteristic decrease in strength.

Ref. 31 also addresses the problematic issue of conducting laboratory tests of unremolded samples of soil with cementation. An alternative method is presented, by which cementation is created in sand through the use of gypsum. The results of unconfined strength tests in samples stabilized with various quantities of gypsum, are presented in Fig. 2.15. The peak strength is achieved at axial deformations in the range of 0.5%, wherein a sharp decrease in strength is mainly observable in samples with high cementation. The higher the cementation the more elastic linear behavior observable along a greater part of the loading curve before peak strength is achieved. The material in which the cementation was carried out was obtained from a source which exhibits natural cementation, and the bonding agent was added to the remolded soil brought from the site. The authors consider that the addition of appropriate quantities of cement material can create a material which will constitute a good simulation of the natural material, with far lower preparation costs and avoiding the great variability which exists in natural soils.

Fig. 2.16 derived from Ref. 32 presents the development of the cohesive and frictional strength factors in granular soils with natural cementation along the Israeli coastal plain. It seems that the cohesive factor in the soil is maximally expressed in deformations lower than 1%, while the expression of the frictional factor is more gradual. The total peak strength of the soil in these tests was obtained under shear deformations of 2-5%. Similar results were obtained by other sources, such as Ref. 30, in soils which received additional cementation. Hence, it is obvious that the lower the soil confinement pressure (as in the case of a wheel moving over the surface of the soil) the lower the deformations at which peak strength is obtained and the faster the decrease in strength following the peak, as a consequence of the relatively small influence of the frictional factor.

2.5 Mathematical Formulation of Strength Changes in Potentially Remoldable Soils.

A number of attempts have been made to model the relationship between shear deformations and strength in brittle soils. Ref. 6 surveys these attempts and a summary is given here. All references up to the end of this section are taken from Ref. 6.

The first attempt to mathematically describe the behavior of strain softening soils was made by M.G. Bekker who proposed the following equation:

$$S/S_{max} = \frac{\exp[(-K_2+\sqrt{K_2-1})K_1 j] - \exp[(-K_2-\sqrt{K_2-1})K_1 j]}{\exp[(-K_2+\sqrt{K_2-1})K_1 j_0] - \exp[(-K_2-\sqrt{K_2-1})K_1 j_0]} \qquad [2.3]$$

where,

- S is the shear stress at a specific point j.
- S_{max} is the maximum stress.
- j is the deformation.
- j_0 is the deformation for S_{max}.
- K_1, K_2 empirical constants.

This equation defines the soil's strength, relative to the peak shear strength, as function of the deformation, the deformation at peak strength and some empirical constants.

The drawbacks of this method are the difficulties involved in finding the empirical K values, as well as the fact that for $j \gg j_0$, the value of S approaches zero, which is, in fact, false.

Oida (1979) proposed a different equation to describe the behavior of the stress-deformation correlations:

$$S/S_{max} = K_r \left[1 - \frac{\sqrt{1-K_r}\,[1+(\sqrt{1-K_r}-1)/K_r]^{j/K_w}}{\sqrt{1-K_r}\,(1-2/K_r)+2/Kr-2} \right] \times \qquad [2.4]$$

$$[1 - [1+(\sqrt{1-K_r}-1)/K_r]^{j/K_w}]$$

where,

K_r is the value S_r/S_{max}, which is the ratio of the residual shear strength to the maximum shear strength.

K_w is the shear deformation, where $S=S_{max}$. (K_w equal to j_Θ in the previous equation).

This function fulfills the following requirements:

1. $S=S_{max}$ for $j=K_w$.
2. $dS/dj = 0$ for $j=K_w$. (The peak point is an extremum point).
3. $S - S_r$ where $j>>K_w$.

The K_w value in the equation is relatively easy to determine as it is the deformation at which the maximum shear strength is obtained. In cases where the S_r, the residual strength, can be determined, it is also possible to determine $K_r=S_r/S_{max}$ and hence the entire strength function. In cases where the S_r value cannot be derived, it is possible to iteratively derive the value of K_r on the basis of the values of a number of points on the stress-deformation curve obtained from the experimental data. This method is rather complicated. The behavior of this equation with the progression of the deformation for the various K_r values is seen in Figure 2.17a.

Another equation was proposed by Wong (1983):

$$S/S_{max} = K_r[1 + [1/(K_r(1-1/e))-1] \times \exp(1-j/K_w)]x[1-\exp(-j/K_w)] \qquad [2.5]$$

Where the definitions of the terms are as in Oida's equation. This equation is easier to calculate, but does not precisely fit the requirements for the first derivation which equals zero at $j=K_w$. (See

Fig. 2.17b). Experiments conducted with Loam have shown a relatively good correlation between the last two models and soil behavior. See Fig. 2.18 (Ref. 6).

The most important problem in the behavior of the proposed equations is that for the given ratio between the residual shear strength and the maximum shear strength, there is only a single functional relation between the deformation and the strength and it is difficult to express variations in the rate of strength change. As can been seen in Fig. 2.18, the soil behaves in a more brittle way than anticipated by both models.

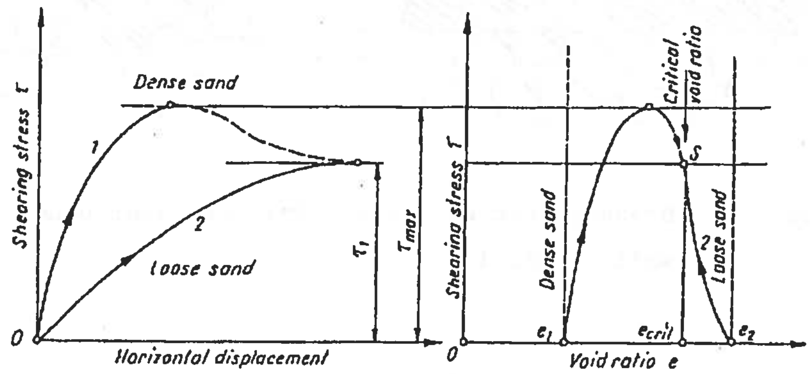


Figure 2.1 Shear tests with loose and dense sand. (Ref. 1)

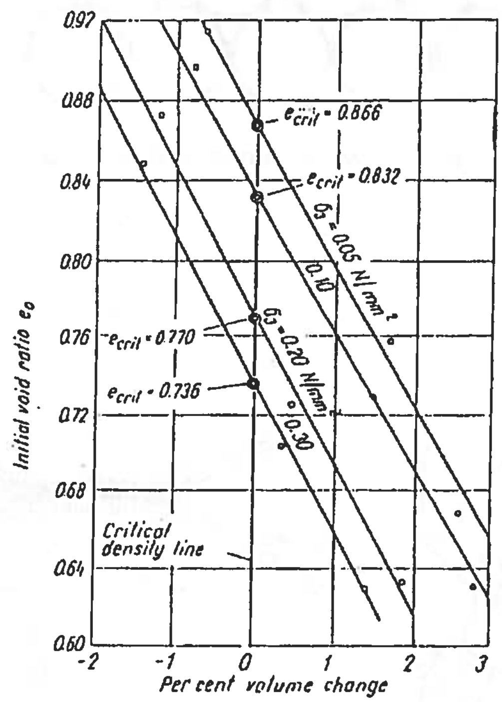


Figure 2.2 Example on the relationship between the shear confinement pressure and the obtained critical void ratio. (Ref. 1)

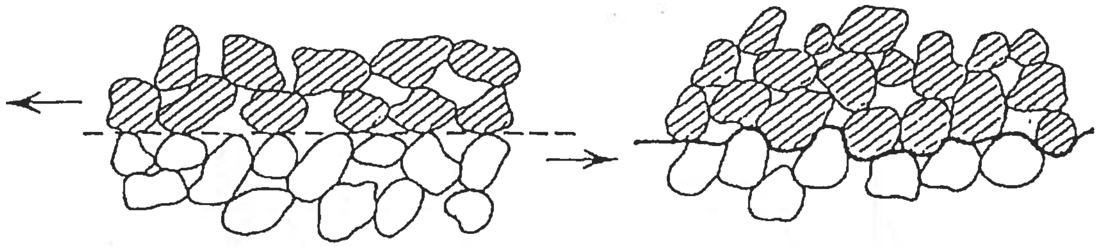


Figure 2.3 Breakage process of the intergranular bonds in granular soil. (Ref. 1).

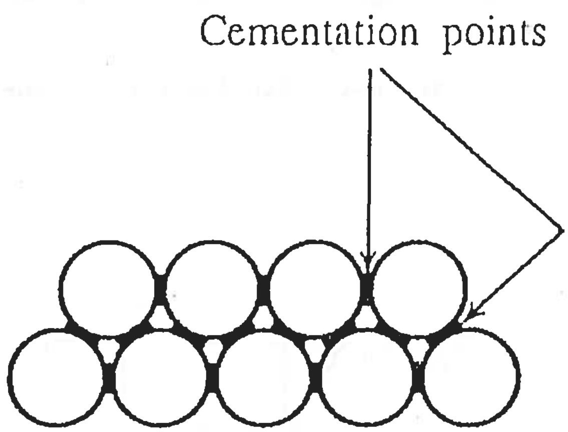


Figure 2.4 Location of the cementation points in granular soils.

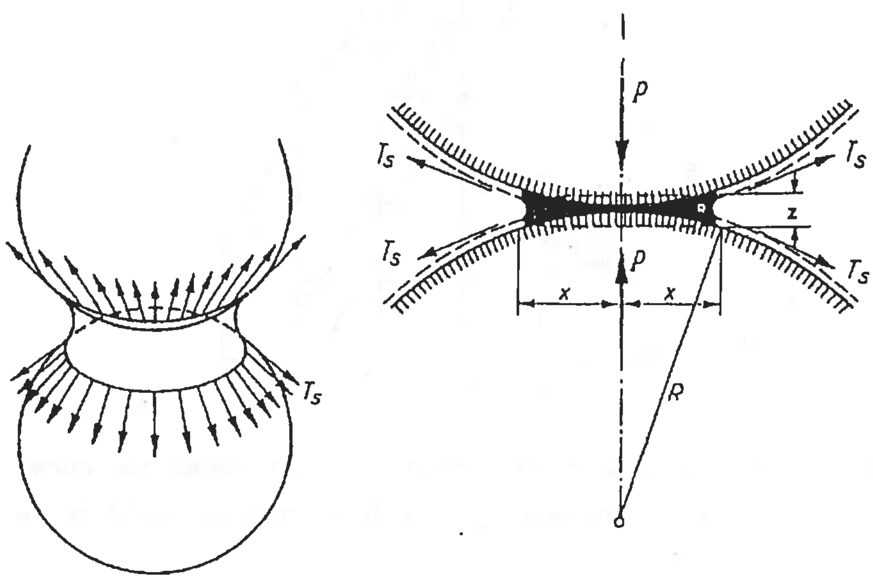


Figure 2.5 Creation of apparent cohesion in granular soils as a result of capillary water pressure. (Ref. 1).

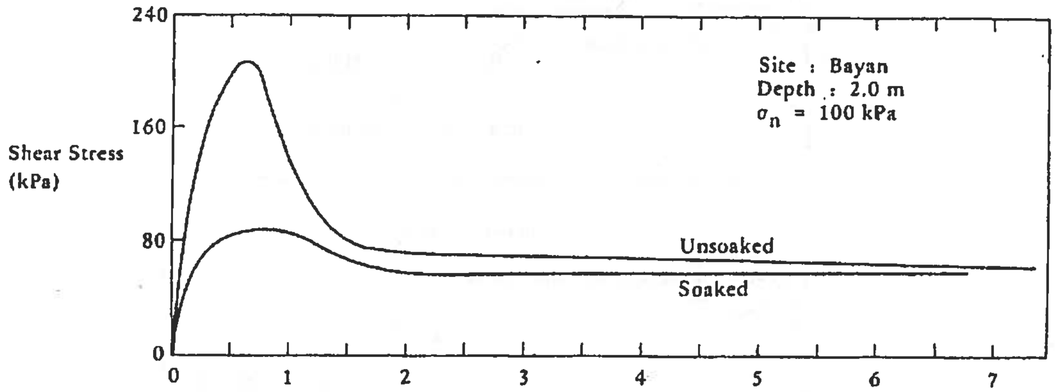
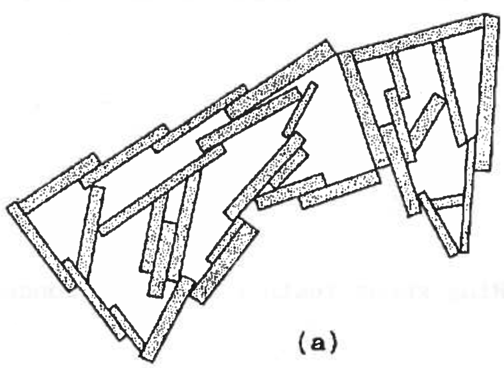


Figure 2.6 **A shear stress-deformation curve for granular soil with natural cementation (Ref.23).**



(a)

(b)

Figure 2.7 **Flocculation and dispersion in clay soils: (a) flocculated structure; (b) dispersed structure. (Ref. 20).**

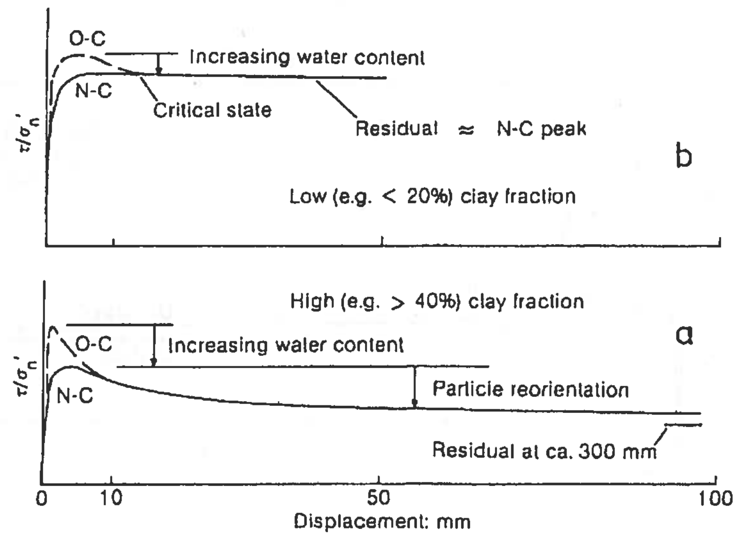


Figure 2.8 Diagrammatic stress displacement curves (Ref. 10).

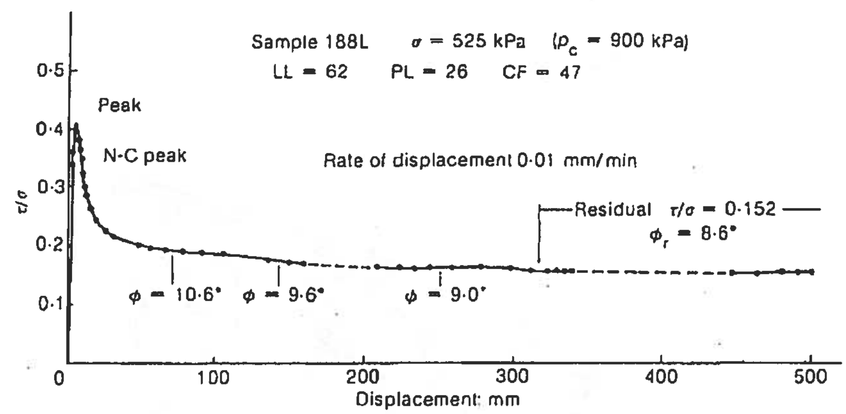


Figure 2.9 Ring shear tests results of bonded clay. (Ref. 10).

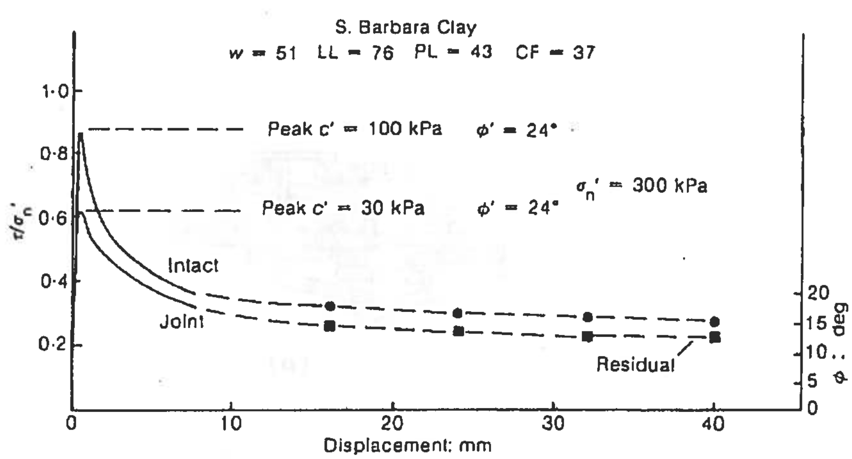
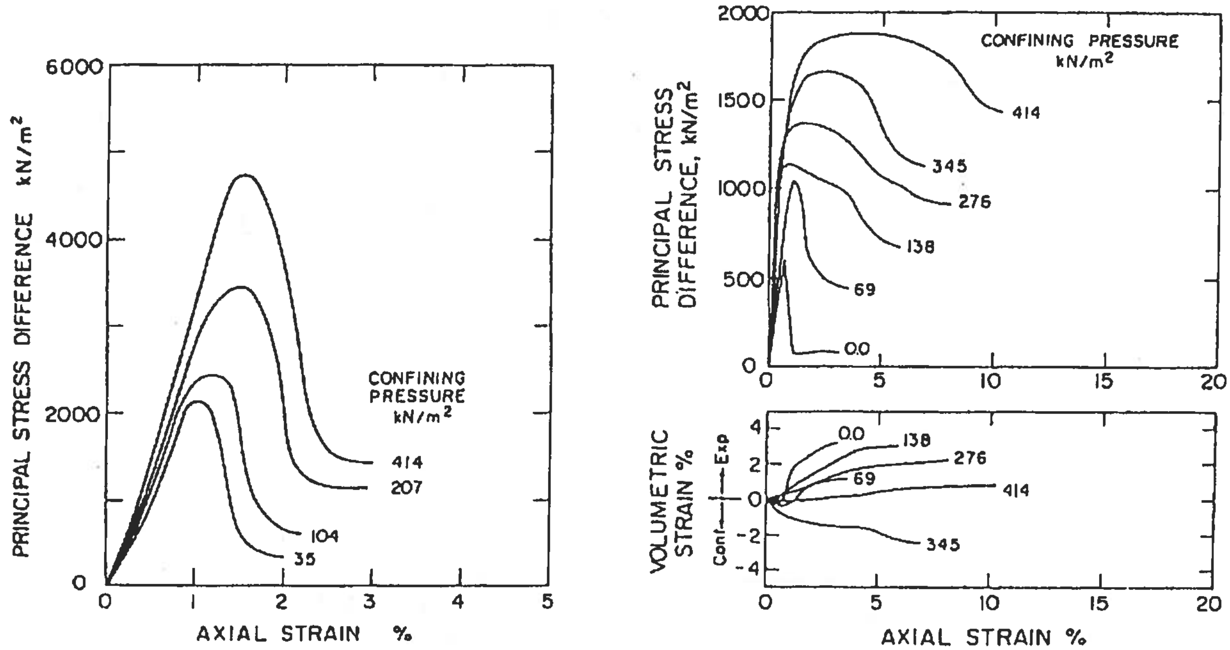


Figure 2.10 Reversal shear box tests on intact clay (Ref. 10).



a. b.

Figure 2.11 a. Stress-stain curves in soil with high natural cementation (Ref. 22).
b. Stress-strain curves in soil with medium natural cementation (Ref. 22).

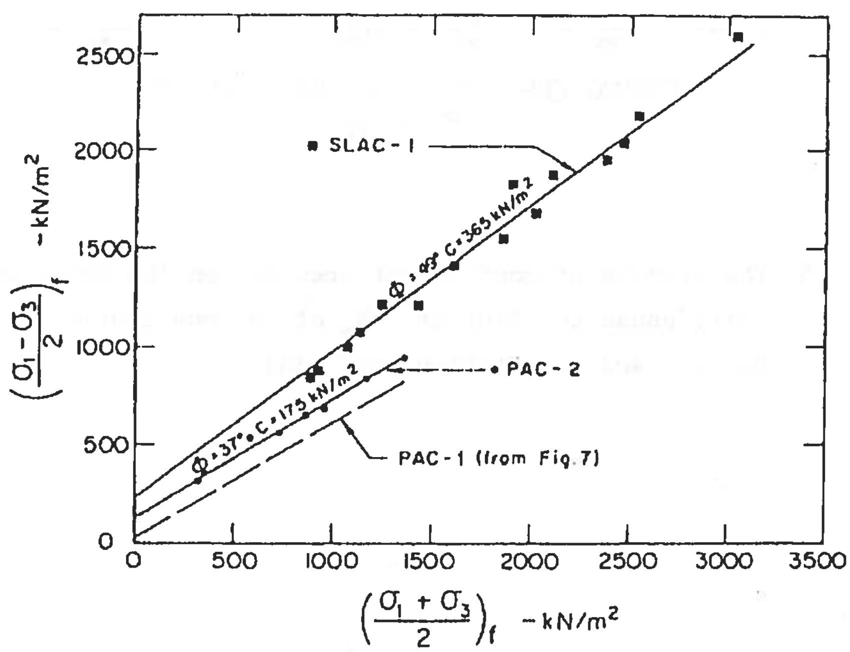


Figure 2.12 Failure envelopes and peak strength parameters in soils with high and medium natural cementation. (Ref. 22)

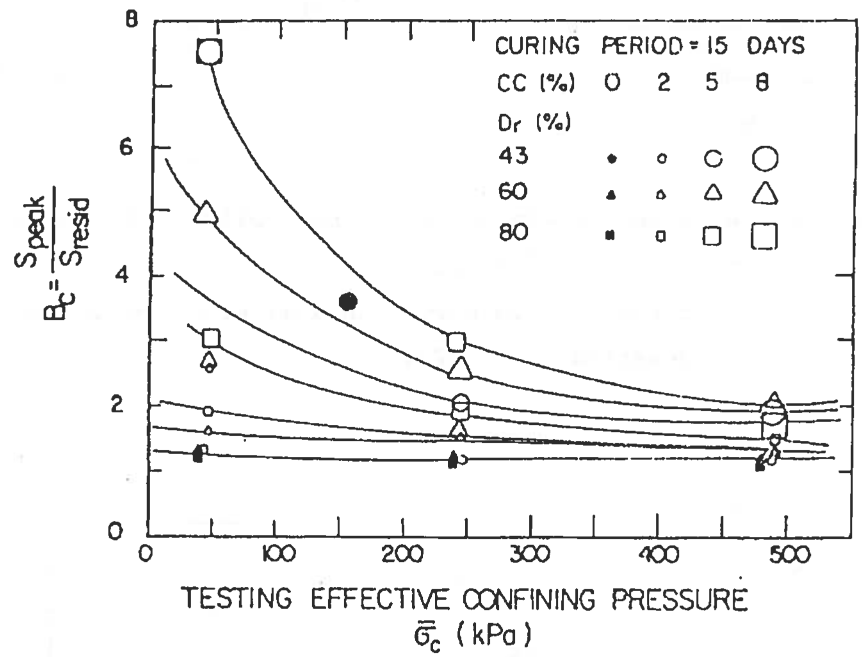


Figure 2.13 **The effects of confinement pressure on the value of the brittleness coefficient - B_c at various levels of relative density and cementation (ref. 30).**

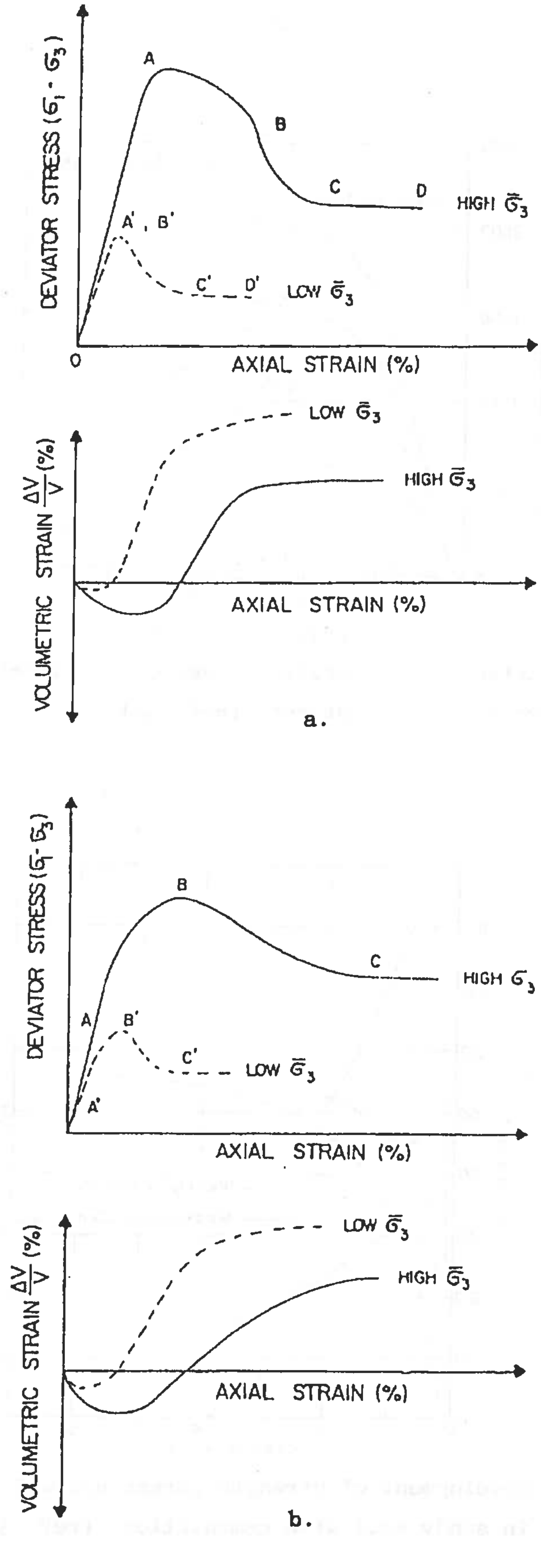


Figure 2.14 a. Strain stress behavior in high cemented sand.
b. Strain stress behavior in weak cemented sand. (Ref.30).

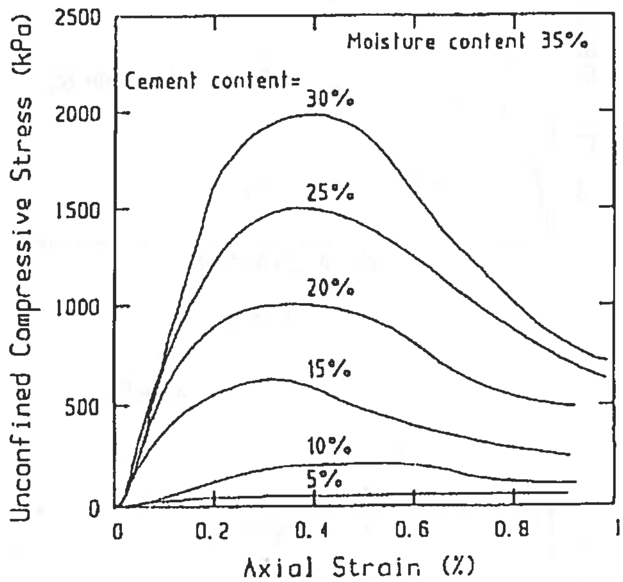


Figure 2.15 **Effect of cementation level on the development of the soil's strain stress. (Ref. 31).**

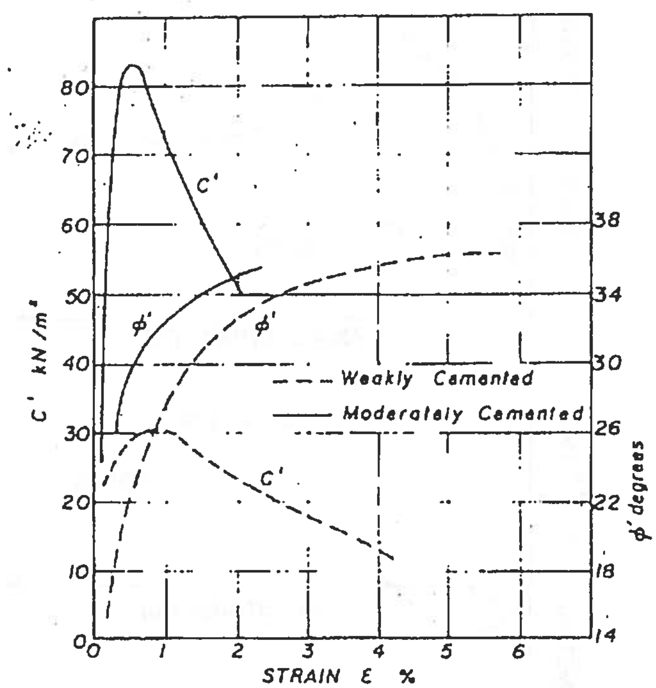


Figure 2.16 **Development of strength parameters with increasing strain in sandy soil with cementation. (ref. 32).**

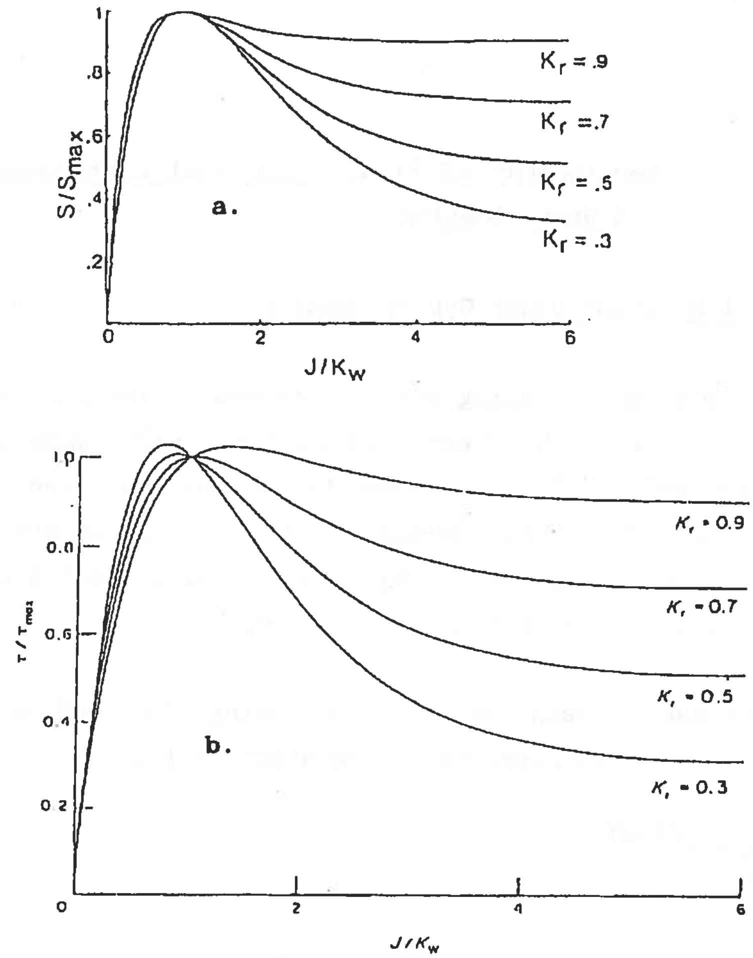


Figure 2.17 Behavior of various equations describing stress-deformation relations in "brittle" soils (Ref. 6).
a. Oida equation.
b. Wong equation.

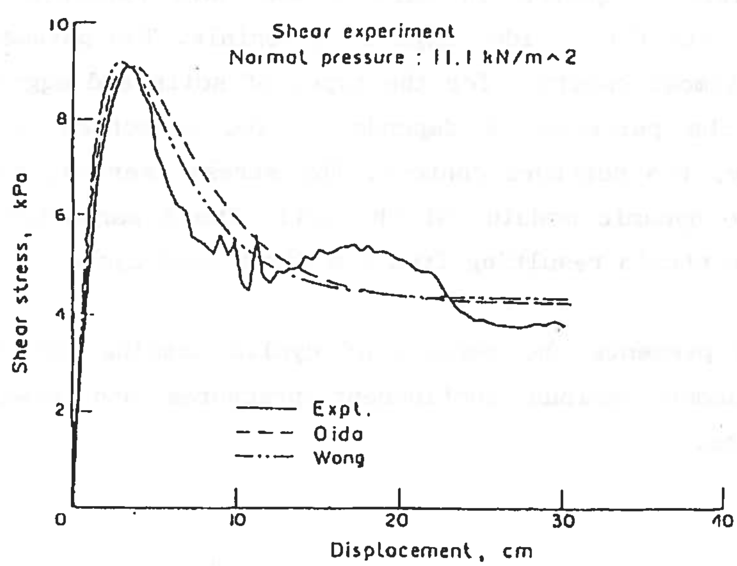


Figure 2.18 Behavior of "brittle" soils during shear. Comparison between predicted and experimental data (Ref. 6).

Chapter 3: Development of Strains and Soil Disturbance under Cyclic and Wheel Loading

3.1 Soil Behavior Under Cyclic Loading

The issue of accumulating plastic strains in the soil as a consequence of cyclic loading has been studied for a wide range of granular and cohesive soils. Ref. 2 presents a comprehensive survey of the literature and an investigation of the plastic strain characteristics in clayey and sandy soils subjected to cyclic loading at a low loading amplitude relative to the soil's strength.

The residual strain accumulating during the loading cycles, called dynamic creep, is described by Equation (3.1).

$$\epsilon_p = A \times N^{(1-m)} \qquad [3.1]$$

where,

ϵ_p is the plastic strain.
N is the number of loading cycles.
A,m are rutting constants.

This equation in different variations has been proposed by additional investigators quoted in Ref. 2 who have examined and showed its suitability for a wide range of materials. The parameter m was found to be almost constant for the types of soils and aggregates examined, while the parameter A depends on the structure of the soil, the density, the moisture content, the stress exerted, the soil strength and the dynamic modulus of the soil. The A parameter is in fact the plastic strain resulting from the first load cycle.

Ref. 9 presents the results of cyclic loading experiments in sandy soil under various confinement pressures and changing deviatoric stresses.

The correlation defined by Equation 3.2 has been discovered to be a good descriptor of the phenomena of strain accumulation with repeated cycles.

$$\epsilon_p = a + b \ln N, \qquad [3.2]$$

where,

ϵ_p is the plastic strain;
N is the number of loadings;
a, b are empirical constants.

Fig. 3.1 graphically describes the results of one of a series of tests and presents the correlation between the plastic strain and the number of cycles for various deviatoric stresses.

The logarithmic relation given in equation [3.1] and the semi-logarithmic one in equation [3.2], present the two most accepted formulas relating plastic strain to the number of load applications. These relations were also adopted for the practical prediction of rutting in conventional pavements under traffic load. (See, for example ref. 16).

In all the researches noted above, the stress level exerted on the samples was far lower than the stress required to achieve failure. When higher loads, closer to the load required to failure, were applied on the samples, a significant increase in the rate of plastic strain was noted, deviating from the expected according to the above mentioned equations. (See Ref. 15 and Ref. 2).

The fact that high cyclic stress levels were hardly investigated, can be ascribed to the situation wherein both in the area of soil mechanics and in the area of conventional pavement design, the loading of the soil occurs in the low stress range in accordance with safety factors coefficients, so that the stress in the soil rarely exceeds 30%-40% of the soil's shear strength.

Some attempts have been made to correlate between the rutting of unsurfaced pavements and the number of load applications.

Trials wherein clay soil was loaded with repeated vehicle coverages, as reported in Ref. 13, yielded the following correlation:

$$r = 0.00107n^{0.5} \times d \times N_c^{-2.6} \qquad [3.3]$$

where,

- r is the rutting depth (in inches).
- d is the external diameter of the unloaded wheel (in inches).
- n is the number of wheel loading cycles.
- N_c is the Clay Mobility Number, which constitutes the parameter which takes into account the effect of the geometric dimensions of the wheel, the load on the wheel and the soil strength and which is calculated according to:

$$N_c = \frac{AI \times b \times d}{W} \times \frac{1}{1 - (\frac{\delta}{h})^2} \times \frac{1}{1 + \frac{b}{2d}}$$

where,

- AI is the airfield index.
- b the width of the unloaded wheel (inches).
- h Height of the wheel carcass (inches).
- δ The difference between the height of the wheel carcass in the unloaded state and in the loaded state (inches).
- W The wheel load in pounds.

The fit between the results of the experiment with a vehicle and the above equation, is seen in Fig. 3.2 for 2, 6 and 10 coverages (the figure is reproduced from Ref. 13).
Similar experiments conducted by Ref. 13 with C-130 aircraft wheels and C-5A aircraft wheels, yielded the following correlation.

$$r = 0.001d \times N_c^{-2.27} n^{0.22} \qquad [3.4]$$

Where the variables are defined as in the previous equation.

Equations (3.3) and (3.4) were determined on the basis of a relatively small number of experiments and in clayey soil only, so that it is hard to estimate their ability to describe a wider range of soils, or even the same clay which was tested, under varying conditions of moisture, loading, number of coverages, etc. At the same time, there is an evident similarity between Equation (3.3) and Equation (3.1) which relates to the strains in the soil and which seems better founded from the point of view of the quantity of experiments and reports presented in the literature.

Where rigidly structured remoldable soils are concerned, plastic strains during cyclic loading under maximum load will be very low. Cyclic loading of the material may lead to fatigue failure and breakage of the strengthened structure, decrease in strength and the formation of large plastic strains.

Ref. 28 has investigated the issue of the behavior of cement stabilized soil under cyclic loading with load values close to the monotonous failure load. Fig. 3.3 presents the results obtained for the predicted number of cycles to failure at various levels of stress for a number of types of soils with a rather high level of cementation. The results of this research correspond to the classic behavior of material fatigue under cyclic loading.

3.2 Changes in Soil Strength Under Wheel Loading

A number of references in the literature have addressed and reported on the phenomenon of soil strength change as a consequence of the passage of a loading wheel. Ref. 24 presents a discussion of the change in soil characteristics in relation to the wheel pass. The discussion centers on tandem-wheel systems, where the soil characteristics after the pass of the leading wheel must be known in order to analyze the behavior of the rear wheel which follows it.

Table 3.1 (After Ref. 24) presents estimates of the phenomena of strength changes in various soils in the rutting path as a consequence of the passage of the leading tandem wheel. The data in the table are no more than general estimates based on experiments conducted in various soils with the movement of a rigid wheel, and in which the soil strength was measured by means of a cone penetrometer.

Figures 3.4 and 3.5 show the distribution of soil strength before and after the passage of the wheel. Loose sandy soil showed a clear tendency to strengthen, probably due to its compression, as compared to loam, where a weakening of the soil was evident after the passage of the wheel. The big differences in soil strength before and after the passage of the wheel, demonstrate the importance of addressing this point.

The behavior of the soil in the rutting lane is generally obtained by combining two factors with different effects on the soil strength. On the one hand the soil in the confined area tends, in many cases, to compact, and hence an increase in the density and shear strength values, while on the other hand, phenomena of decreases in strength, such as the destruction of the cementation, the breaking of the inter-granular bonds, the breaking of the capillary ties, etc., appear as a consequence of remolding of the soil's initial structure.

The soil strength after the passage of the wheel will be determined by the relative dominance of these two general factors of influence. The high loading rate of the wheel dictates a situation of undrained loading which does not allow changes in volume in many soils, even those having a relatively coarse grading. In light of this, in soils close to saturation (category C in Table 3.1), the compaction effect is not felt. In this case one can therefore expect phenomena of strength decrease as a consequence of the disturbance of the original soil structure. When the soil's rate of saturation is lower, the compaction of the soil is usually possible and one can therefore obtain higher strength values after the passing of the wheel,

Table 3.1: Changes in Soil Strength Under a Passing Wheel. (After Ref. 24).

Soil Type	Category	Degree of Saturation	Effect on Soil Propetries
SEDIMENTARY	a	0-30%	Cohesionless loose soils gain in strength due to compaction. Very dense cohesionless soils (unlikely to occur in nature) lose strength due to reworking. Friable silt and clay are likely to gain strength. Loess is likely to lose its cohesion.
	b	30-90%	Partially saturated soils generally gain in strength due to compaction. The most gain is attained at optimum moisture content. Information on compaction (available in civil soils engineering literature) is applicable.
	c	90-100%	Highly saturated soils are unlikely to gain strength since for practical purposes the compacting effect is negligible. Clays with structure are likely to experience collapse or reorientation of particles and lose strength in reworking.
RESIDUAL	d	0-100%	Residual soils usually exhibit some cohesion due to their structure. If wheel load breaks the structure (lead wheel sinks significantly), cohesive strength is likely to be lost in reworking.

especially in those cases where the original density was low and the structural strength (which was disrupted by shearing) was also low.

Ref. 21 presents the results of experiments conducted by repeated passes of a pneumatic wheel over sandy soils (only about 5% passing sieve #200) which contained cementation. The soil's strength values as a function of the number of wheel passes (see Fig. 3.6 a,b) demonstrates the above. The first pass of the wheel caused a 60% decrease in the soil's strength due to the breaking of the inter-granular cementation. Repeated passes of the wheel led to a gradual increase in the soil strength as a result of continued compaction of the sandy soil. Following the first pass, the cementation seems to have disappeared and the only factor which effected the change in strength was the compaction factor.

In wet soils (no more details are given as to the moisture content of the soil), the decrease in soil strength is much less significant (only about 25%). Ref. 18 also reports actual aircraft landing exercises which were carried out in sandy areas in which cemented sand with relatively high strength was remolded and loosened by the very first aircraft landings.

3.3 Stress Paths Under a Moving Loading Wheel.

Application under laboratory conditions of a stress path similar to that occurring in the field is in many cases essential for a better understanding of soil shearing processes.

Ref. 17 presents an analysis of the state of stresses at a point under the pavement surface, resulting from the passage of a loading wheel. In order to facilitate the calculations, the soil has been assumed to be an elastic homogeneous infinite medium, and an equally spread load P_0, with a width of 2a, is applied to the soil. According to the Boussinesq equations for planar stresses, the following stress components are obtained (see Fig. 3.7):

$$\sigma_V = \frac{P_0}{\pi} [\theta_0 + \sin \theta_0 \cos(\theta_1 + \theta_2)] \qquad [3.5]$$

$$\sigma_H = \frac{P_0}{\pi} [\theta_0 - \sin \theta_0 \cos(\theta_1 + \theta_2)] \qquad [3.6]$$

$$\sigma_{VH} = \frac{P_0}{\pi} \sin \theta_0 \sin(\theta_1 + \theta_2)] \qquad [3.7]$$

where,

σ_V - Normal vertical stress
σ_H - Normal hoizontal stress
σ_{VH} - Shear stress on the horizontal plane.

$\theta_0 = \theta_2 - \theta_1$

and,

θ_2, θ_1 is the angle (in radians) between the perpendicular and the line connecting each of the ends of the loading surface to the point being tested in the soil.

When the values of equations [3.5] - [3.7] are known, it is possible to calculate the principal stresses

$$\left.\begin{matrix}\sigma_1\\ \sigma_3\end{matrix}\right\} = \frac{\sigma_V + \sigma_H}{2} \pm \sqrt{\left(\frac{\sigma_V + \sigma_H}{2}\right)^2 + t\tau_{VH}^2} = \frac{P_0}{\pi} (\theta_0 \pm \sin\theta_0) \qquad [3.8]$$

And the direction of the major principal stress relative to the vertical axis will be derived from:

$$tg2\beta = \frac{2\tau_{VH}}{\sigma_V + \sigma_H} = tg\ (\theta_1 + \theta_2) \qquad [3.9]$$

Accordingly, one can calculate the distribution of the various stress elements along the horizontal plane at various distances from the loaded area (see Fig. 3.8 presented in Ref. 17). In the case of a moving wheel, every point at a certain depth will undergo the entire stress path as depicted in Fig. 3.8 during the passage of the wheel.

Accordingly, when it is required to simulate the stresses acting on a point of soil under the load of a moving wheels, one must consider the simultaneous changes which take place in the size and the direction of the principal stresses at that point.

Cyclic shear tests on loose sand were conducted (see Ref. 17) using two different stress paths. The first was the conventional stress path of the triaxial compression test and the second was a stress path which emulates the stress induced by a passing wheel (as depicted in Fig. 3.8). The results (see Fig. 3.9) show a great difference in soil reaction due to different stress paths. (Residual volumetric strain was up to 5 times greater in the latter stress path).

Actual wheel loading on unsurfaced soil is much more complicated than assumed in the above analysis and the soil reaction is not purely elastic. However, the results brought by Ref. 17 emphasized the importance of comparing as closely as possible the stress paths pertaining in the field in order to better understand the soil behavior under the given condition.

The above issue is important when considering laboratory experiments intended to investigate soil behavior under wheel loads. Ref. 2 also addresses this issue, but on account of an absence of appropriate laboratory equipment, the experiments where carried out in a triaxial shear device, with alteration of the confinement pressure during loading. The desirable solution in this case is the simultaneous change of both the confinement pressure and the principal stress direction during the loading process.

3.4 Approach of Present Design Methods to the Remolding Phenomenon.

All existing design and prediction methods require, as part of the design process, that the value representing the soil strength be obtained in order to allow the prediction of the number of cycles to failure. The usual design process rarely includes any direct discussion of the soil remolding phenomenon. In those cases when the soil's remolded strength is estimated (using field testing methods to be detailed in the next section, and only in soils with a significant cohesive component), use was made of one of the two following values:

a. The remolded strength is used as the soil strength for design purposes. (See Ref.4 and a literature survey in Ref. 5.).

b. The average value of the natural strength and the remolded strength is taken as the soil strength for design purposes. (For example, Ref. 13, Ref. 14 mentioned in Ref. 12).

The first method is obviously very conservative. In certain cases, such as Ref. 21, this evaluation method seems to approximate reality as the structural strength resulting from soil cementation actually seems to break after a single wheel pass. In other cases, where the loading of the unremolded soil is not so intensive (the loading ratio, i.e., the ratio of the wheel load to the bearing capacity of the soil, is small), the remolding process may continue over a large number of cycles, so that the estimation of the design strength according to the first method may yield overly conservative and unrealistic results. According to the second method, in which the average of the strength values (remolded and unremolded) was taken as the design value, the prediction may predict a far greater number of cycles than should be predicted in the case of Ref. 21, and thus lead to dangerous design.

Common to both methods is the simplicity of their application using existing design methods, but it is possible to clearly see their inability to predict the number of cycles to destruction in many actual situations.

3.5 Present Methods used for Determination of the Remolding Potential

The nature of an unsurfaced runway's construction and operation usually greatly limits the possiblities of carrying out orderly laboratory tests, and for this reason it is necessary to develop appropriate in-situ testing tools which will allow a quick evaluation of the soil's strength change potential.

Ref. 4 details the elements and mode of usage of a standard instrument for testing soil remolding potentials. Fig. 3.10 presents a photograph of the elements comprising the testing system. The kit includes the following main components:

1. Steel sample cylinder, 2" in diameter and 9" in length (marked A in Fig. 3.10.)
2. A basis for the sample cylinder (marked B).
3 A 2.5 lb. drop hammer (marked C).

The main testing procedures are:

1. Extraction of an undisturbed sample from the soil by means of a soil traficability sampler.

2. Transfer of the soil sample directly to the remolding cylinder which is placed firmly on its base.

3. Using the penetrometer in order to evaluate the strength of the undisturbed soil within the cylinder. The soil strength will be determined according to the mean penetrometer readings at depths of 0-4" (5 readings). In the event that the soil strength is too high and it is not possible to penetrate down to a depth of 4", the missing values will be registered as a strength of $300 lb/in^2$.

4. Placing the foot of the drop hammer on the upper part of the soil sample in the cylinder and carrying out 100 hammer blows (The height of the weight's drop is 12").

4a. For mixed soils in which the granular element is relatively predominant, the remolding process is not done with a drop hammer but by covering the upper part of the sample cylinder with a tight cover in direct contact with the soil, and allowing the test cylinder to drop 25 times from a height of 6" onto a firm surface.

5. After completion of the remolding procedure, the penetrometer is used again to determine the remolded soil strength.
The remolding index is calculated by the following equation:

$$RI = \frac{CI_R}{CI_0} \qquad [3.10]$$

where,

R_I is the remolding index.

CI_0, CI_R are the cone penetrometer readings in the cylinder before and after remolding, respectively.

and the remolded soil strength value is calculated by:

$$RCI = CI \times RI \qquad [3.11]$$

where,

RCI is the Rating Cone Index which is assumed to approximate the strength of the remolded soil.

CI is the initial strength of the soil as measured in the field (not in the cylinder).

RI is the Remolding Index calculated by [3.10].

An estimation of the expected Remolding Index range according to Ref. 4 for various groups of soils is:

1. Fat inorganic clay soils 0.75-1.35
2. Mixtures of clay and granular materials, thin and medium clays, silty clays 0.45-0.75
3. Mixture of silts and granular materials, silty soils, thin silty clays and organic soils 0.25-0.85

The remolding test described above suffers from a number of drawbacks which, in our estimation, greatly limit its ability to foresee soil behavior under vehicle movement.

1. The remolding procedure of the soil sample takes place in a small cylinder wherein the soil is in a confined state.
   The confinement of the soil greatly limits the possible shear displacement among the soil particles. Most of the added strength agents display strong and brittle behavior, and some shear strain is needed in order for them to disappear. The remolding procedure in the confined cylinder does not seem to give the soil enough 'freedom' to develop shear strains (as it has in the field), so it might well be that in some cases only part of the remolding phenomenon will be detected. The above effect is even more pronounced in dense granular soils where shear is accompanied by dilatation. The confined state of the soil in the sample cylinder allows practically no dilation so shear and remolding processes are not free to occur.

2. Like the soil remolding process, the process of inserting the penetrometer into the testing cylinder is carried out when the soil is in a confined state and its free movement is very limited. This state is completely different from the actual in-situ conditions. An attempt to avoid biasing the remolding test results was made by using only the ratio of the strength values taken in the cylinder to be multiplied by the actual initial strength value measured in the field (equation 3.11). However, the very different conditions which prevail during the in-situ penetrometer testing and the testing in the cylinder, are not only expressed in the absolute value of the strength results obtained, but also in the relative change in strength due to the remolding. The remolding and weakening of the soil will be expressed in a different way within the narrowly dimensioned testing cylinder and in the field where the soil is much less confined.

3. Cohesionless granualr soils (or granular soils with a small amount of cohesion), cannot be tested by the proposed instrument because of the inability to extract an undisturbed sample. This limitation prevents the possibility of identifying change in the strength of soils of this type.

The above points emphasize the drawbacks of the remolding testing method and its limitations as an instrument for the evaluation of soil strength change following shear by a loading wheel.

The above remolding test may provide a relatively good indication in those cases where it is necessary to identify clay sensitivity. In cases of sensitive clay, it is possible that the blow process itself (stage 4 of the testing process), despite the soil shear limitations as detailed above, will lead to the remolding of the clay structure and to a significant decrease in strength.
In less sensitive soils, where larger shear strains are required in order to reveal the remolding potential, the above mentioned testing process may yield overly high remolded strength values.

Ref. 21 reports a number of simple field attempts to evaluate the remolding potential of sandy soil containing cementation. Attempts were made to apply a 6" by 2" shear vane, but these attempts failed, mostly because of soil remolding as a consequence of the penetration of the shear vane itself into the soil.

Attempts to simulate the remolding of the soil by a vehicle wheel were made by simple methods using a bayonet and a penetrometer. The bayonet was used to remold a soil segment about 10cm. in diameter through loosening. After loosening, the soil surface was stamped with the foot until it returned to its orginal level before the remolding. The researchers considered the results obtained by this method unsatisfactory, mostly because of the small diameter of the remolded area which led to strong lateral confinement of the remolded segment and overly high strength values (despite the fact that the diameter of the remolded segment was twice that of the cylinder in the above mentioned remolding test).

The method which produced the best results for predicting the soil strength after the passage of vehicles, was that which used a shovel and a penetrometer. The shovel was used to dig a pit, 40cm in diameter and 30cm deep. The dug-out material was loosened and then replaced and compacted (with the foot) down to its original level. Penetrometer tests were carried out before and after the remolding, and the researchers found a good correlation between the strength ratio during the test and the strength ratio obtained before and after the passage of vehicles.

In the context of a discussion which took place at the same conference, Turnbull (Ref. 21), addressed this issue. He assessed the proposed method as logical and simple to perform, yet at the same time raised the idea of inroducing a thin-walled cylinder of 6-8" diameter into the soil, within which the remolding will be carried out by ramming with a rod. The strength tests will be carried out before and after the remolding in order to evaluate the potential strength change.

Though the system proposed by Ref. 21 is seemingly primitive, and lacks the usual hallmarks of a controlled test, it nevertheless offers a number of advantages:

a. The strength test before and after the remolding are carried out under field conditions without the problems of extracting soil samples and the problem of confinement by the test cylinder.

b. The test is very simple and requires a minimum of time and effort.

c. The test can be carried out in almost any type of soil.

3.6 Preliminary Observations in Granular Remoldable Soils under Wheel Loading.

Ref 19., which is a part of phase IV of this research presented the results of landing exercises of C-130 aircraft at two sites in Israel. The local soils at the two sites lack plasticity and have been defined as A-1-b (site T) and A-2-4 (Site S) according to the AASHTO method. Despite the very high initial strength values obtained by means of DCP tests (CBR values in the range of 40%-100% at site T and in the range of 40%-80% at site S) extremely significant rutting was measured, far beyond that predicted by accepted design nomograms.

Fig. 3.4 presents an example of the rutting results obtained at site T along the runway following 6 and 10 cycles of C-130 landings. (The B value denotes the height difference from the original surface to the bottom of the rut, while the A value denotes height difference from the earth mounds at the sides of the rut to the bottom of the rut). Strength tests conducted at the center of the rut after the completion of each serie of landings did not reveal any significant change in the soil's strength (Ref. 19). Strength tests conducted after the completion of the loading cycles revealed loose material at the bottom of the rut to a depth of a number of centimeters, whose strength had not been taken into consideration. Later tests at site T clearly revealed the formation of a loose upper soil layer as a consequence of the passage of the aircraft.

It seems that despite the non-plastic character of the soil, the cementation created in the unremolded soil contributed to the very high strength prior to the beginning of landings. Initial laboratory tests of the local material taken from site T produced the following results:

a. Remolded material with a moisture content of 4% was compacted inside a 37x18x10 cm. mould, to a dry density of about 1770 kg/m^3, and dried to a moisture content of about 2.0%. Average strength values of about 2-3 kg/cm^2. were obtained by menas of a pocket penetrometer. A laboratory moving wheel load test with contact pressure of about 5kg/cm^2 was carried out on the soil sample (see photograph adn more details on the laboratory device in chapter 5) The sample failed (about 1.5 cm. rutting of the 15 cm. diameter rigid wheel) following about 80 wheel passes, wherein the soil in the rutting area and to the sides of the rut became loose (no more than 0.5 kg/cm^2).

b. The same soil sample was then leveled, compacted to its original height and moistened until a moisture content of 14% had been achieved. There was no change in the dry density of the material. The soil was dried in an oven at a temperature of 60°C. Strength tests conducted with a pocket penetrometer at moisture contents of 3% and less, indicated a very high strength, over 5kg/cm. (the upper limit of the penetrometer's measuring capacity). A moving wheel test conducted with identical loads to those utilized in the previous case, was carried out when the sample moisture content was 1.8%. Even after about 600 loading cycles no destruction could be seen in the sample. A great deal of effort was required in order to disintegrate the sample and the resultant soil lumps were far stronger than those of the previous sample.

These preliminary data may explain the high initial strength of the soil at site T whose natural moisture content was in the range of 1%-2%. The in-soil drying process after the moistening of the rains, produces inter-granular cohesion, probably through capillary pressures or minute quantities of cementing factors. A similar phenomenon seems to have been observed in site S as well, and may have taken place at many other sites, mainly in arid and sem-arid areas where the natural humidity is low.

As mentioned previously, these strength factors are transient ones and are annulled after the soil has been loaded to the point of failure. This phenomenon was clearly seen both at the site and in the laboratory in the form of loose and weak soil which was formed over the original soil. Section 4.3 below presents details of initial experimental results obtained from induced remolding of the local soil at site T (Table 4.1). The great difference in soil strength values before and after remolding, emphasizes the need to take this phenomenon into consideration in the design methods of unsurfaced runways.

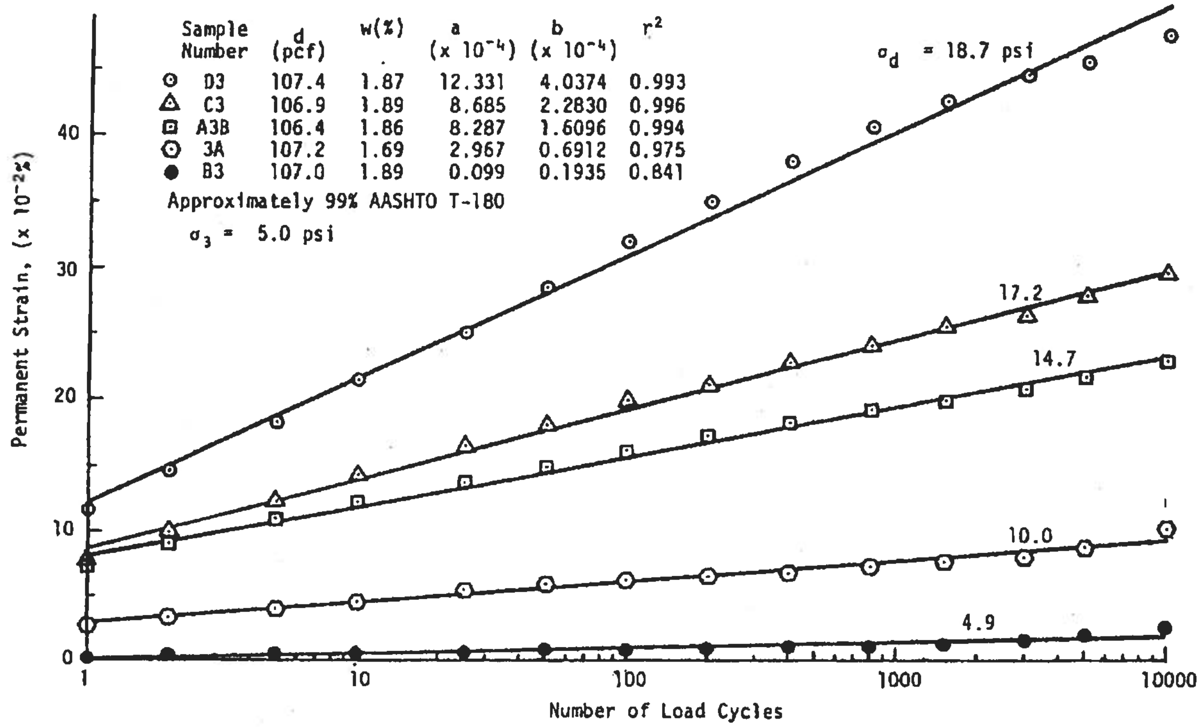


Figure 3.1 Correlation between residual plastic strain and the number of cycles for different conditions of principal stress differences. (Carried out under a confinement pressure of 5 psi). (Ref. 9).

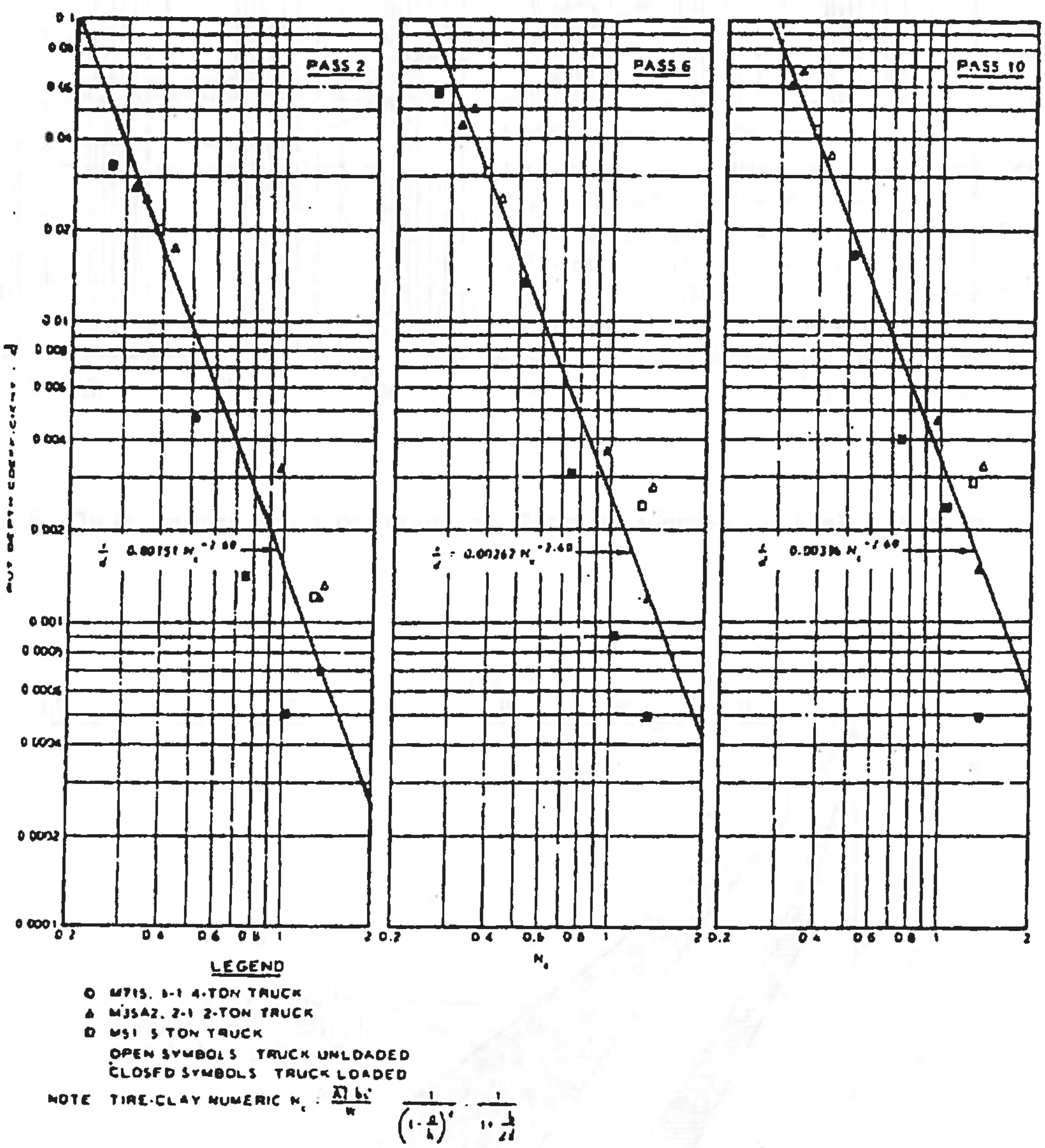


Figure 3.2 Relation between repetitive wheel rutting and N_c (Reported by Ref. 13).

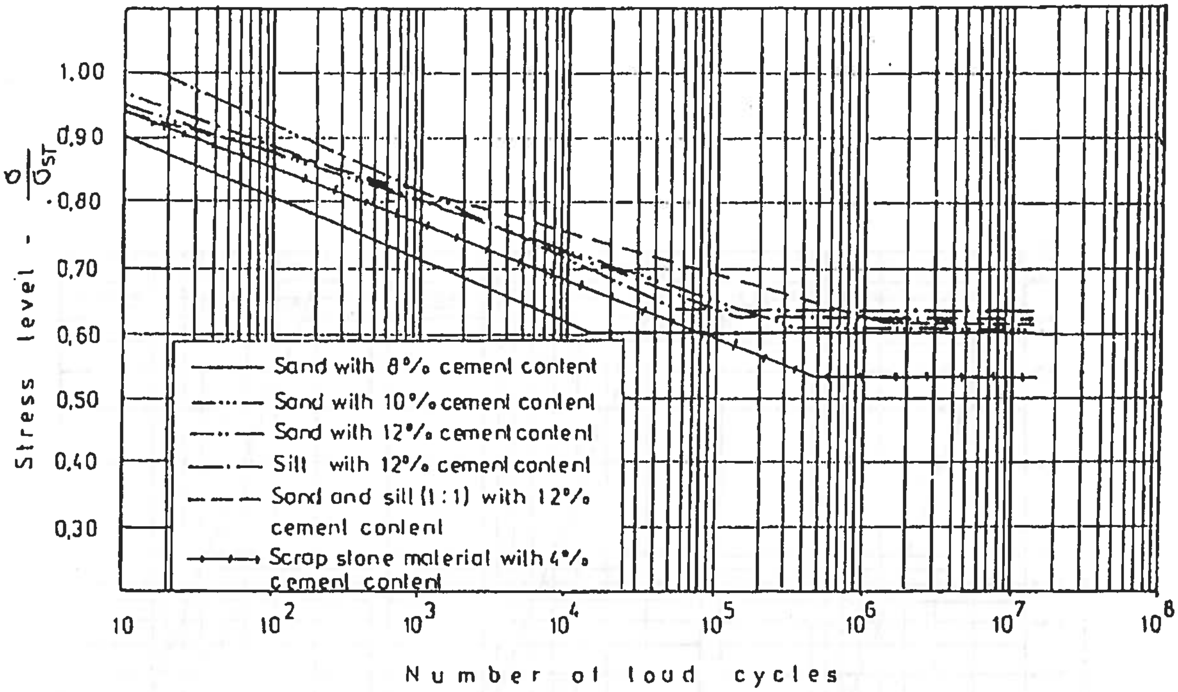


Figure 3.3 Fatigue curves in soils stabilized with cement (Ref.28).

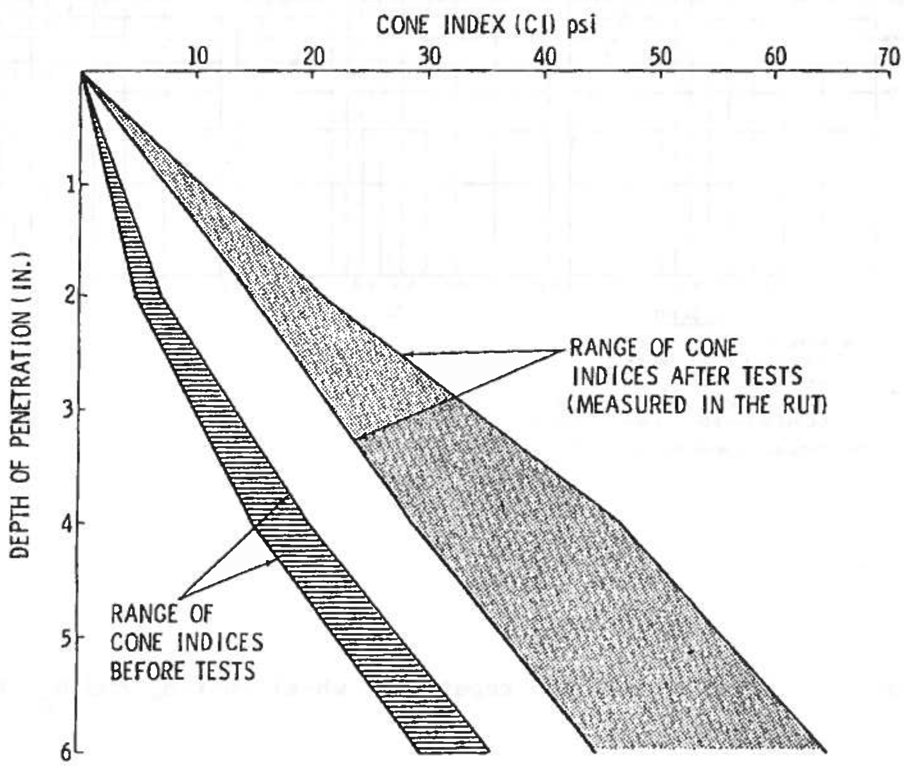


Figure 3.4 Cone penetration resistance in loose sand before and after passage of wheel.

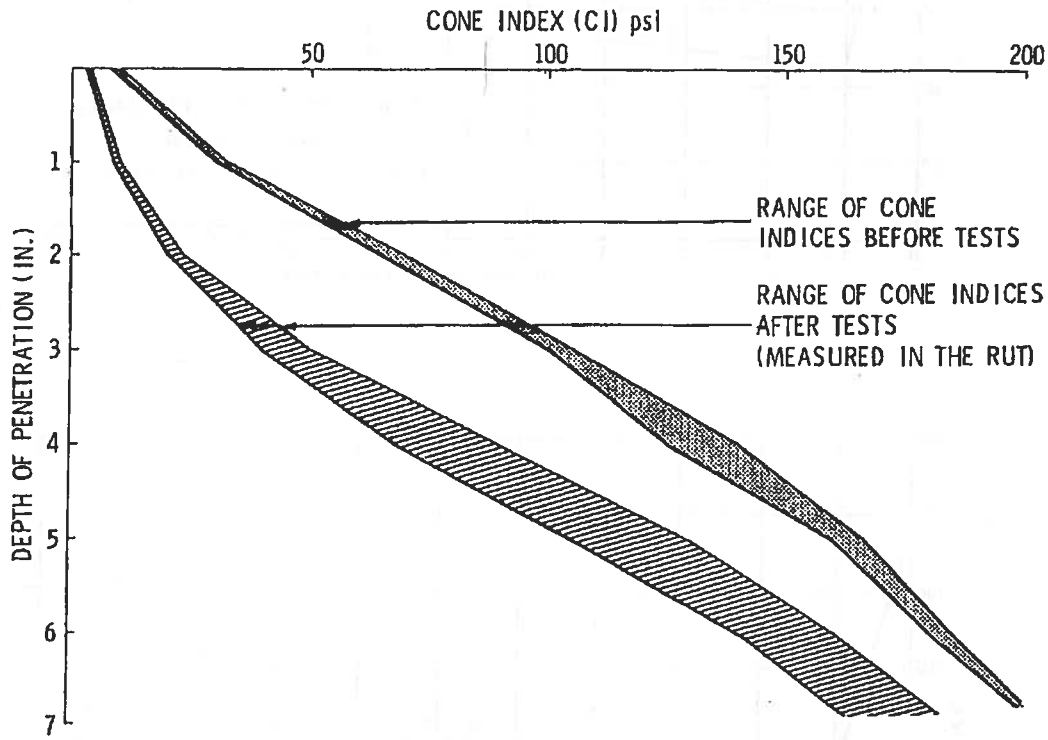
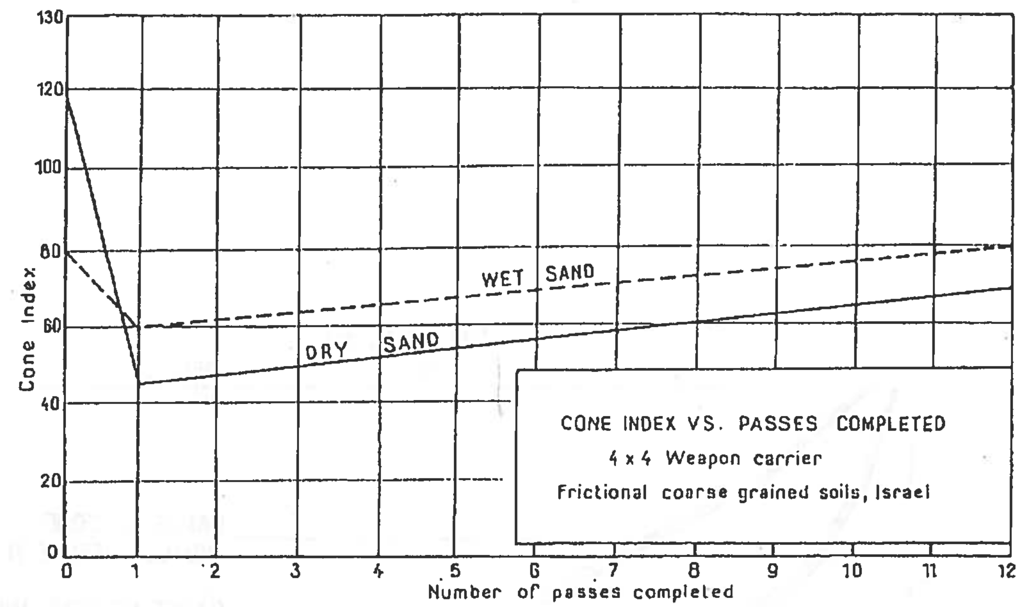


Figure 3.5 **Cone penetration resistance in loam soil before and after passage of wheel.**



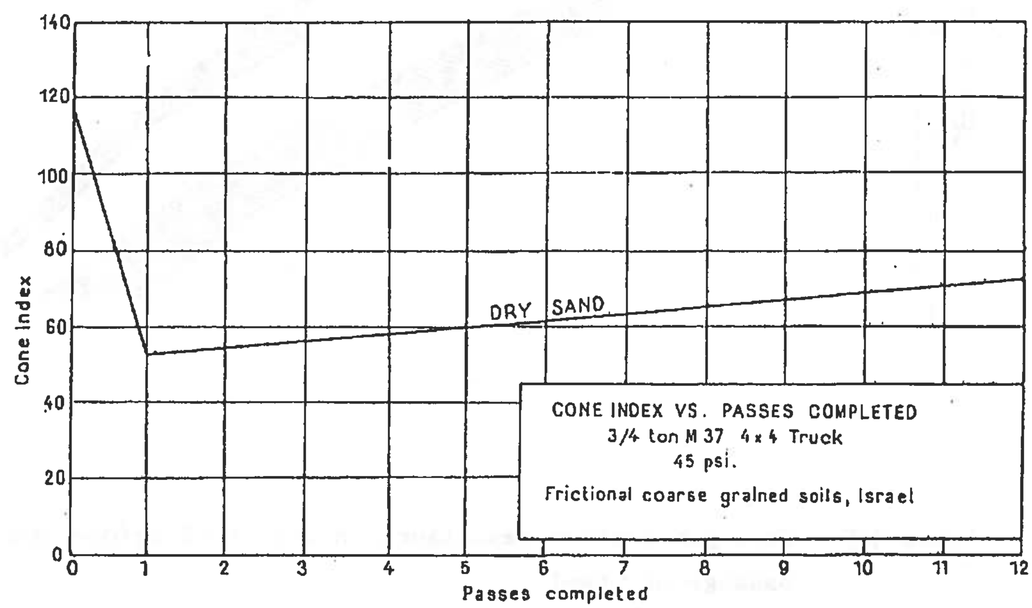


Figure 3.6 Cone Index Vs. passes completed (Ref. 21).

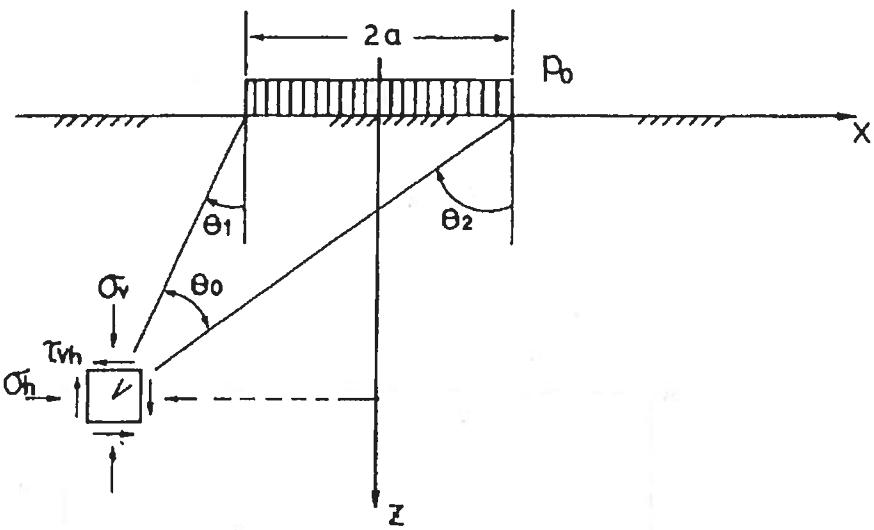


Figure 3.7 **Stress components of a point in homogeneous elastic soil under surface loading (Ref. 32).**

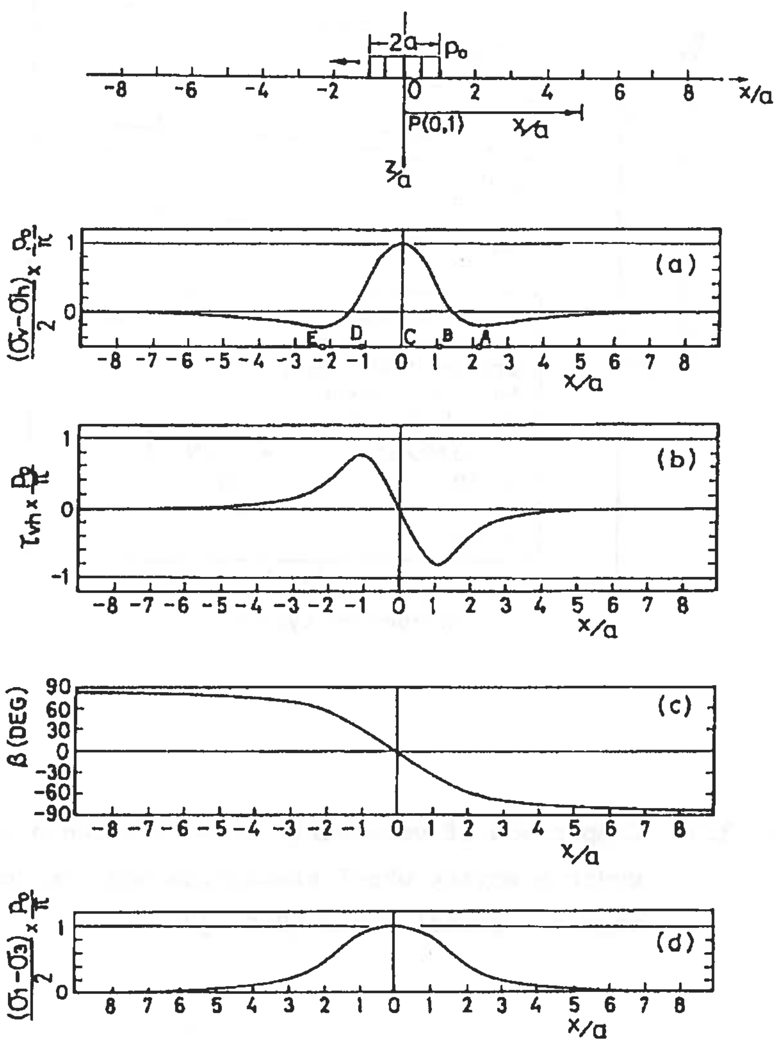


Figure 3.8 **Lateral distribution of a number of stress components at a point in homogeneous elastic soil under a surface loading of 2a diameter (Ref. 32).**

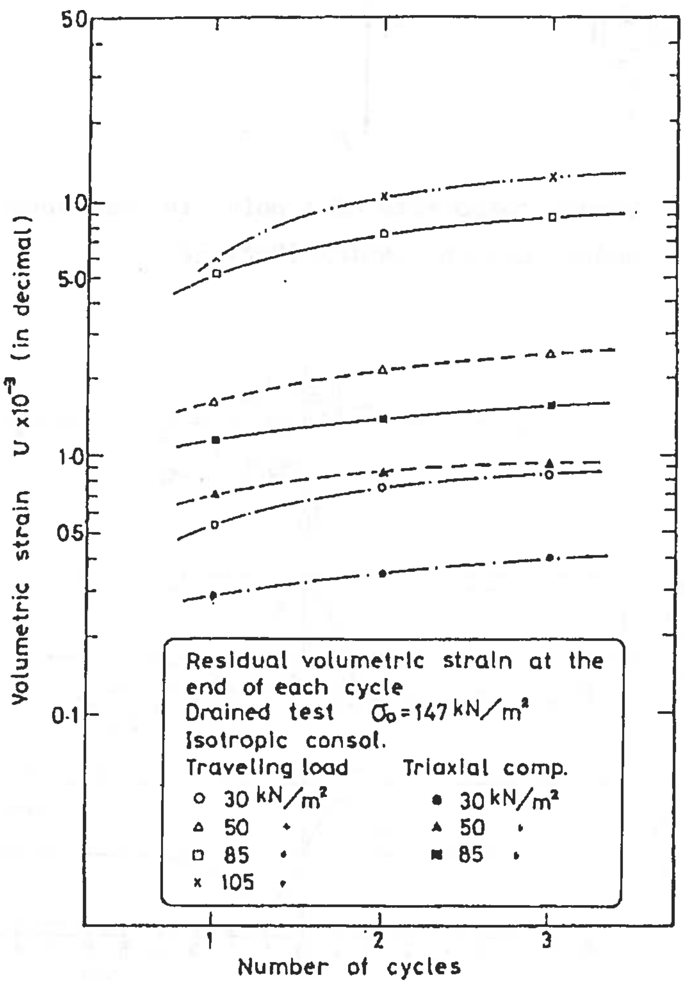


Figure 3.9 **Comparison of volumetric strains between samples tested under a moving wheel simulation and samples tested under regular spatial shear (Ref. 32).**

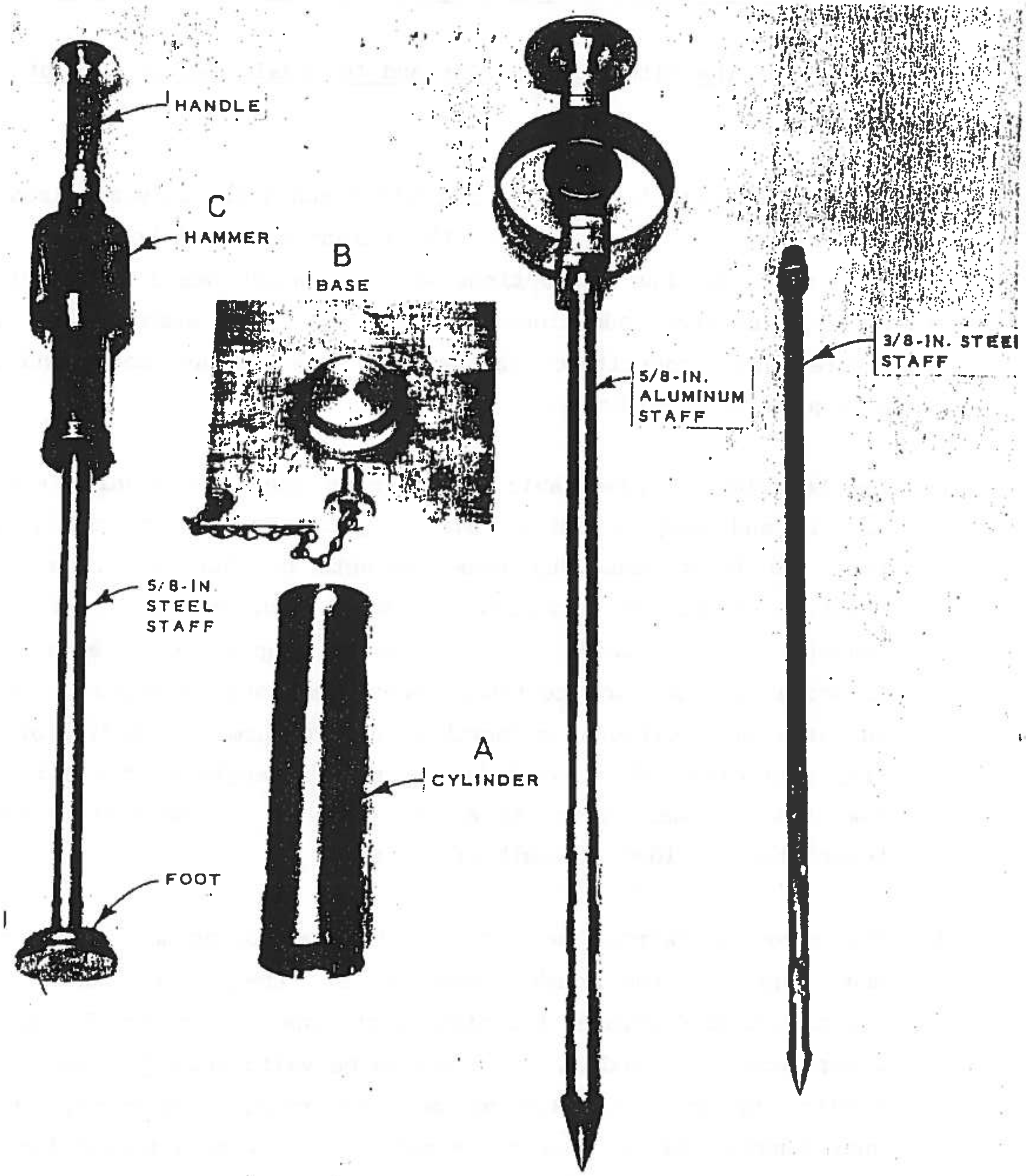


Figure 3.10 Remolding test equipment.

Chapter 4: The Proposed Model for Describing the Behavior of Potentially Remoldable Soils under Aircraft Movements.

4.1 Summary of the Literature Survey and the Basic Assumptions of the Model.

The literature revue detailed in chapters 2 and 3 can be summarized by a number of main points. Some of the points detailed below are, at present, no more than assumptions which, though based on existing literature, require additional validation. The summary of the literature survey constitutes the basis of the proposed model and the research plan detailed below.

a. The behavior of remoldable soils under shear is usually mostly elastic and quite rigid in the pre-peak zone (where the stress level is lower than the peak strength of the soil), and the plastic strains are very small. This behavior is most apparent in granular soils which contain some degree of cementation (especially under low confining pressures) and in cohesive soils in which the additional strength is due to chemical bonding of the clay particles. After reaching the peak strength of the soil (at low strain values) there is a fast decrease in the soil strength toward the remolded strength of the soil.

b. The stresses exerted on the soil by the moving wheel are high and approach the peak strength of the soil, while the relationships developed for plastic strains vs. number of loadings (equations [3.1] and [3.2]) claim to be valid only for low stress levels (up to about 40% of peak strength). Therefore, it is questionable whether the above relations can be utilized for the solution of the problem which is the concern of the present research.

c. Because of the relatively rigid structure of the unremolded soil, it seems that for the problem at hand, deformations and hence rutting of the soil, are mainly due to post peak deformations in the already remolded soil. Plastic strains in the unremolded soil are relatively negligible.

d. Remolding of the soil may be caused by cyclic loading of the soil at levels lower than the peak strength of the unremolded soil. In these cases, S-N fatigue functions may be built which relate the stress level to the number of load applications until failure, i.e. the remolding of the soil.

e. In saturated soils, the fast loading rate will dictate a condition of undrained shear in most cases and the soil density will remain stable during the shear. However, in unsaturated soils, density changes will usually take place in the sheared soil in most cases. The soil density after shear is a function of the type of soil and the confinement pressure of the soil during the shear process.

f. In granular soil exhibiting cementation, it was found that the additional strength of the unremolded soil is mainly cohesive in nature. Destruction of the additional strength through the remolding of the soil, led to the disappearance of the cohesive part of strength (the C factor in the Mohr-Coulomb equation), while the frictional part, the internal angle of friction, remained practically constant.

g. Additional strength factors in the soil, such as cementation, negative pore pressures and structural strength resulting from chemical bonds in clayey soils, are transient factors related to the shear processes in the soil. Strengthening processes (such as thixotropic processes) are not taken into consideration and the loss of strength is irreversible in the context of pavement performance prediction.

h. Observations carried out at a landing site where the soil was a potentially remoldable granular soil, indicated that the rutting process was carried out through the remolding of the upper part of the soil, and the loss of the soil's cementational element. Beneath the loose upper layer, the soil retained its original strength.

i. The passage of a moving wheel over the soil creates a stress path of special character which includes changes in the direction of the principal stresses. This phenomenon influences the reaction of the loaded soil. In order to obtain a reliable prediction of soil behavior through laboratory tests, it is recommended to emulate this stress path as closely as possible.

j. The existing testing methods for evaluating the remolding potential in soils have many limitations and can only be carried out in workable cohesive soils.

k. Most of the references dealing with the design of aircraft performance over unsurfaced runways do not consider the issue of soil strength change. In those cases where this issue is addressed, the approach is both simplistic and very limited. No model was found to describe the development of soil strength change under the cyclic loading of a moving wheel.

4.2 Principles of the Proposed Model and Solution Methods.

The literature survey and the observations conducted on potentially remoldable soils under repeated wheel loadings, constitute the basis for the construction of a model describing the behavior of these soils. The aim of the proposed model is to improve the prediction of the soil's strength profile throughout the life of the pavement, and thus help to better estimate the number of cycles until the failure condition is achieved.

Before the beginning of the wheel movement, the initial state of the soil is assumed to be a semi-infinite homogeneous medium. The passage of the wheel causes stresses in the unremolded soil, stresses which gradually decrease with depth. When the stress level of the soil reaches the soil's peak strength, failure results and the soil strength sharply decreases to the soil's remolded strength value. Fig. 4.1A presents a cross-section of the loading wheel and notes the area which has been remolding as a consequence of a single wheel's passage. The actual lateral wander of aircraft movement creates a remolded upper layer which lies on top of the natural soil base (Figure 4.1B). Such a phenomenon was indeed observed during preliminary trials conducted in sandy soil with cementation under the cyclic loading of a laboratory moving wheel device.

The brittle nature of the soil's behavior permits the assumption of a two-layer medium, wherein the interface is characterized by a discontinuity of the strength values rather than the continuous state which would have been expected in a ductile material. According to this assumption, the brittle behavior of the stress-strain function of the potentially remoldable soil, may be approximated by an elastic perfectly plastic stress-strain function which includes a point of discontinuity following the peak point and the decrease of the soil strength to a residual value (see Fig. 4.2)

With repeated loadings, the thickness of the remolded soil layer gradually increases in accordance with the stress levels reaching the unremolded layer. The continued failure of the soil also takes place as consequence of fatigue caused by stress conditions below the peak soil strength. The rate of increase in the upper layer thickness gradually decreases and one may expect it to attain a certain asymptotic value of remolded soil layer thickness over the original soil. It can be seen that the remolded soil layer serves to separate between the wheel and the unremolded layer beneath it, which aids in the distribution of stresses and in decreasing the destruction rate of the unremolded segment.

The thickness of the remolded soil layer according to the proposed two-layer model may depend on a number of major factors:

$$h = F(C, \phi, E_G, VR, \gamma, F, L, n) \qquad [4.1]$$

where,

- C the soil's cohesion (in both the remolded and unremolded state.
- ϕ the internal angle of friction (in both the remolded and unremolded state.
- E_G elastic characteristics of the soil.
- VR soil void ratio.
- γ soil's unit weight.
- F the unremolded soil's fatigue function S-N.
- L characteristics of the loading wheel (geometric data and loading data).
- n number of loading cycles.

In the case of potentially remoldable soils in which the added strength factor is cementation, most of the additional strength seems to manifest itself in increased soil cohesion. The remolding of such soils soils breaks the cohesive factor, while the angle of internal friction remains almost constant. In such a case, the soil's Mohr-Coulomb parameters will be:

$$C_N = \begin{cases} C_N^\circ & \text{- Before remolding} \\ 0 & \text{- After remolding} \end{cases}$$

While, the ϕ value does not change during the remolding process.

The issue of collapsible soils, where the cementation is annulled in a fast process which significantly increases the density and strength, is not included in the present research. During the first stage of the model's analysis, a situation will be assumed in which there are no density changes in the soil during the shear and remolding process, and there is thus no change in the void ratio and the internal angle of friction (according to the results obtained in the literary survey). Later on, an attempt will be made to relax this assumption and to establish void ratio and an internal angle of friction which are more comparable to those formed under in-situ wheel loading conditions. Possible examples of the change in the thickness of the remolded layer h with the progression of loading cycles, according to the two-layer model, can be seen in Fig. 4.3.

Once the function describing the development of the remolded layer thickness has been defined, it is possible to proceed and develop the model in one of the following two main directions:

a. Expansion of the model for the numeric calculation of the accumulated ruttings arising from repeated aircraft passes over a two-layer structure of changing thickness, until the formation of ruts corresponding to the failure criterion. This approach has the essential advantage of being more general and enabling flexibility in the operating conditions of the aircraft (such as lateral wander, loading conditions, wheel geometry, etc.), so as to permit the calculation of the accumulated rutting at each stage up to the failure criterion of the runway. At the same time, the full development of such an approach is a complicated task which requires the construction of new design nomograms which differ from the currently accepted nomograms, and may require the application of new and different field methods in order to measure soil characteristics.

b. Incorporating the results of the two-layer model evaluations in existing design nomograms. This approach has a number of important advantages in that it is less complex than the above described approach, and in that it is based, to a great extent, on existing and well-known methods of field testing and aircraft performance prediction. The main disadvantage of this approach is the adoption of the inherent limitations of existing design methods (such as, for example, lack of consideration for different soil types in the prediction processes, limitations of the penetrometer test in evaluating the behavior of various soils, etc.), which unsolved, continue to influence the quality of the solution.

the main stages according to this approach are:

1. Evaluating the thickness of the upper remolded layer in the two-layer structure, depending on the number of cycles.
2. Determining the weighted strength of the soil as a function of the thickness of the remolded layer and the strength of the various layers. A proposal for the determination of the weighted strength value of a two-layer soil structure, is described by Ref. 25 according to the model presented by Ref. 26 and ref. 27. Fig. 4.4, taken from Ref. 25, presents an initial example of the process of determining the weigthed soil strength in a two-layer structure. The soil strength is determined according to the equation:

$$CI_{equiv} = K \times CI_{LOW} \qquad [4.2]$$

where,

CI_{equiv} is the weighted soil CI value.

CI_{LOW} is the CI value of the weak layer (in our case, the upper layer).

K is a multiplier determined according to Fig. 4.4.

The output of this stage will be a weighted strength function of the soil, depending on the number of loading cycles.

$$CBR_{equiv} = F(CBR_0, CBR_R, h) \qquad [4.3]$$

where,

CBR_{equiv} is the weighted soil strength.

CBR_0 is the soil strength before remolding.

CBR_0 is the strength of the remolded soil layer.

h is the thickness of the remolded soil layer.

Figure 4.5 presents an example of the possible behavior of the soil's weighted strength function with the progression of the loading cycles. It is important to note that the process of the decrease in the soil's strength is not necessarily ended after N cycles (where N is the number of cycles to pavement failure). In many cases, this process is practically ended much earlier than pavement failure and the pavement will continue to function with a decreased strength until the point of failure is reached.

3. The predicted number of cycles to pavement failure will be determined by combining the soil's equivalent strength vs. loading cycles function with the accepted design nomograms which relate the number of cycles to failure with the soil strength. According to most design methods (Ref. 12), the following relation is accepted:

$$N = axCBR^b \qquad [4.4]$$

where,

a,b are empirical parameters related to loading characteristics, wheel geometry, etc.

CBR is the soil strength.

N is the number of cycles to pavement failure.

Use of the Miner Law will permit the desired solution to be obtained:

$$\int_0^N \frac{dn}{axCBR_{equiv}^b} = 1 \qquad [4.5]$$

An important factor in the prediction process is the evaluation of the soil strength after its remolding (the CBR_R value of Fig. 4.5). This issue was only partially handled and will be discussed in section 4.3 below. When the soil strength values are known before and after the remolding of the soil (CBR_0 and CBR_R), the main problem is to determine the rate of change of the equivalent soil strength from its initial value to its remolded value. In any case, the number of cycles to failure will be limited within the range:

$$aCBR^b_R \leq N \leq aCBR^b_0 \qquad [4.6]$$

where the required number of cycles is in the range of lower and upper limits calculated according to the corresponding limits of soil strength given in Fig. 4.5.

At this stage it seems that research efforts should focus on the latter approach, due to its simpler and more practical nature.

4.3 General Outlines of the Proposed Remolding Test Method

In light of the limitations of present methods, an attempt will be made to develop a simple method which will allow the change in soil strength to be identified. The main requirements which this method should fulfil are:

a. The method should be easy to learn and easy to apply, considering the field conditions under which it will usually be applied.

b. The instrument has to be mobile and, if possible, even capable of being carried and used by a single person without requiring a vehicle to transport or operate it.

c. The soil remolding must be carried out under such conditions as will ensure the shear deformations among the soil particles.

d. The remolding process will be carried out in such a way as to allow change in the density of the disturbed soil. The wheel passes over soil which may have been loosened by preceding passes. In order to find out more accurate values of the remolded soil strength, the soil must be allowed to change its density during the remolding process and attain a value close to the critical density which corresponds to the confinement pressure under the loading wheel.

Most of the soils used for aircraft landings in Israel are granular soils and therefore the use of the remolding instrument as presented by Ref. 4 is impossible. As a first stage, it was decided to use the main principles of the simple method proposed by Ref. 21, while making certain changes in the method of placing back the soil after remolding. The proposed method will include the following stages:

1. Carrying out DCP tests within a radius of 1m. in order to determine the initial soil strength value for that area.

2. Digging a 40 cm. diameter and 30cm. deep test pit, taking out the dug material and loosening it with the help of the digging instrument.

3. Returning the dug material into the pit in two layers of about 20cm. each. The soil will be poured freely into pit and each layer will be leveled and compacted by means of a drop hammer. The compaction will be carried out so that the compaction energy is controlled and equally divided over the surface of the pit.

4. Once the compaction is complete, a repeated DCP test will be carried out in the remolded soil. The decrease in strength will be determined as the ratio of the soil strength after remolding to the average of tests before remolding.

This remolding method has the advantages of the method proposed by Ref. 21, with additional control over the energy invested in compacting the soil, and removal of the requirement of returning the soil to its previous level. In the estimation of the researchers, this simple method will make it possible to estimate in a relatively precise way, the remolding potential of the area, without requiring additional instrumentation and based on the standard testing instrument used - the DCP.

An initial attempt to use the proposed remolding testing method was made at site T. A soil area of 40/40cm. was dug to a depth of 15-20cm. at two places along the runway which have not been loosened. The digging was carried out by means of a pick-axe and the soil was loosened and crumbled until all earth lumps disappeared. The earth lumps before the remolding were very firm and it was difficult to crumble them by hand. Following remolding of the structure, the soil was replaced and compacted by means of an 18kg drop hammer with a diameter of about 20cm., which was dropped about 30 times over the remolded surface from a height of about 15cm.

Table 4.1 - Results of Remolding Tests at Site T (7/91)

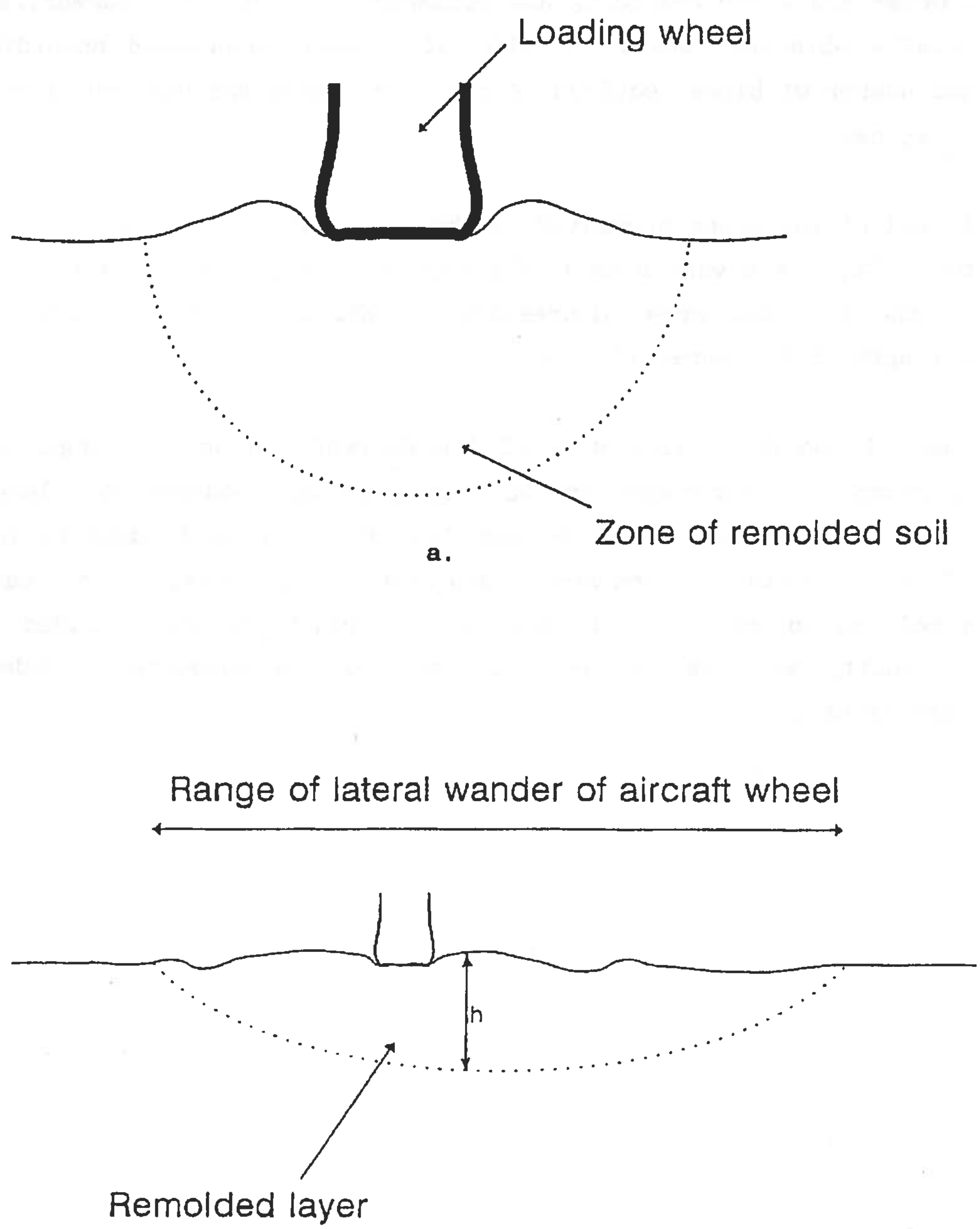
Dry density (kg/m^3)			CBR (%)	
Point No.	Before Remolding	After Remolding	Before Remolding	After Remolding
1.1	2026	1839	47	5.8
2.1	1952	Unmeasured	57	5.8
3	1844	1739	57	2.9
4	1803	1757	46	3.0

Figures 4.6 a, and b. present photographs of the remolded soil surface before the compaction process and following recompaction, as well as the remolding and recompaction equipment. DCP tests and density tests using a nuclear density meter, were carried out before the remolding

process and after remolding and recompaction. Table 4.1 summarizes the results obtained, where the CBR values were calculated according to the number of blows required in order to insert the DCP rod to a depth of 15 cm.

In all of the cases presented in the table, the soil density decreased by 2-10%, but a very drastic decrease took place in the soil strength in the remolded area, decreasing by 90% and more relative to the strength of the unremolded soil.

There is no doubt that some of the decrease in soil strength can be ascribed to the decrease in the density of the loosened soil layer. At the same time, one must consider that the decrease in density is part of the remolding phenomenon and thus, when coming to test the remolding potential, it is desirable to reach (in the remolded layer) a density as close as possible to the one encountered under the loading wheel.



b.

Figure 4.1 **a. Zone of remolded soil under a single wheel path.**
b. Effect of lateral wheel distribution on the creation of a remolded layer.

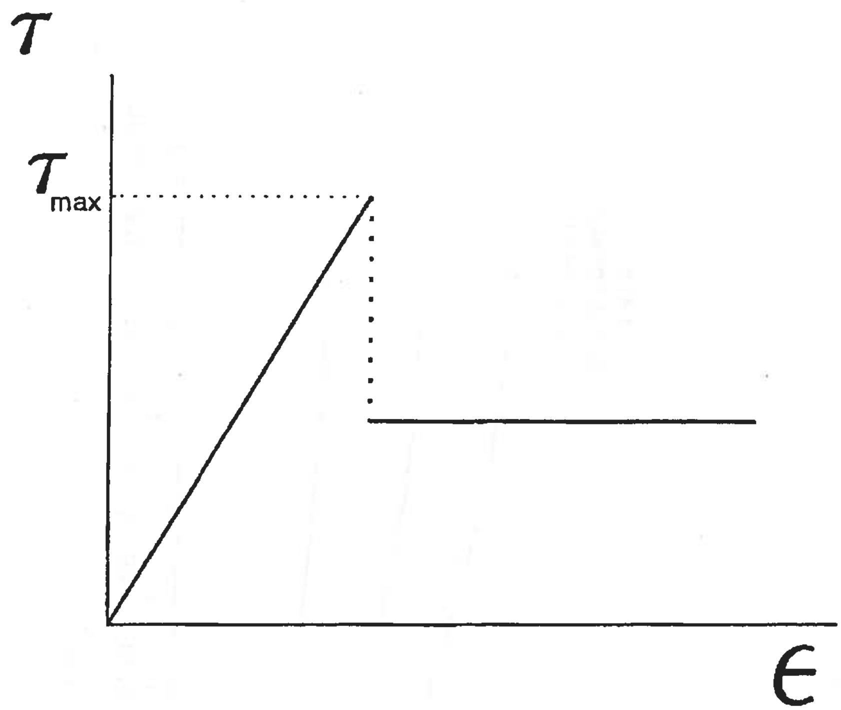


Figure 4.2 Approximation of the Stress-strain function in cementitious granular soil.

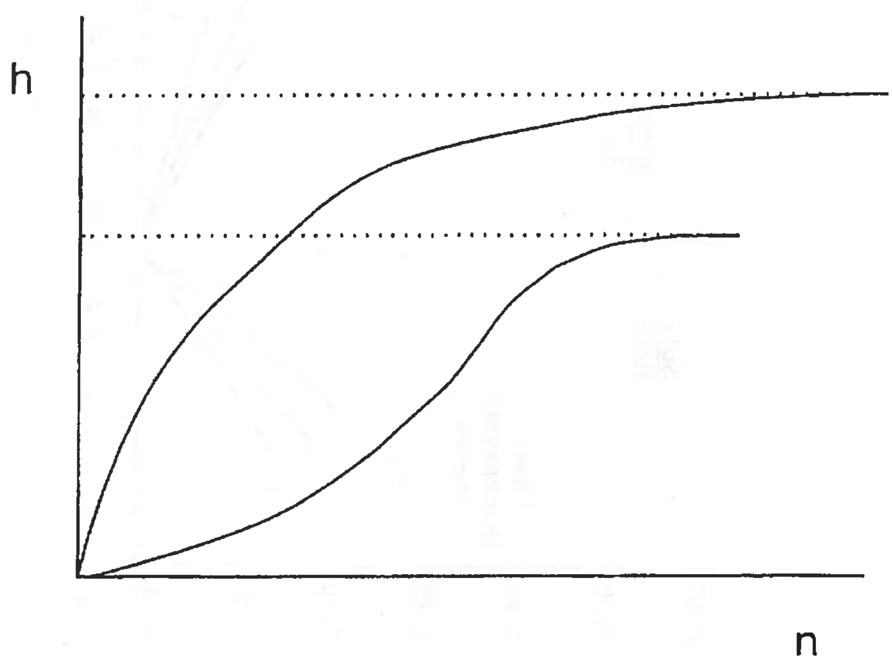


Figure 4.3 Assumed relationship between remolded layer thickness and number of wheel coverages.

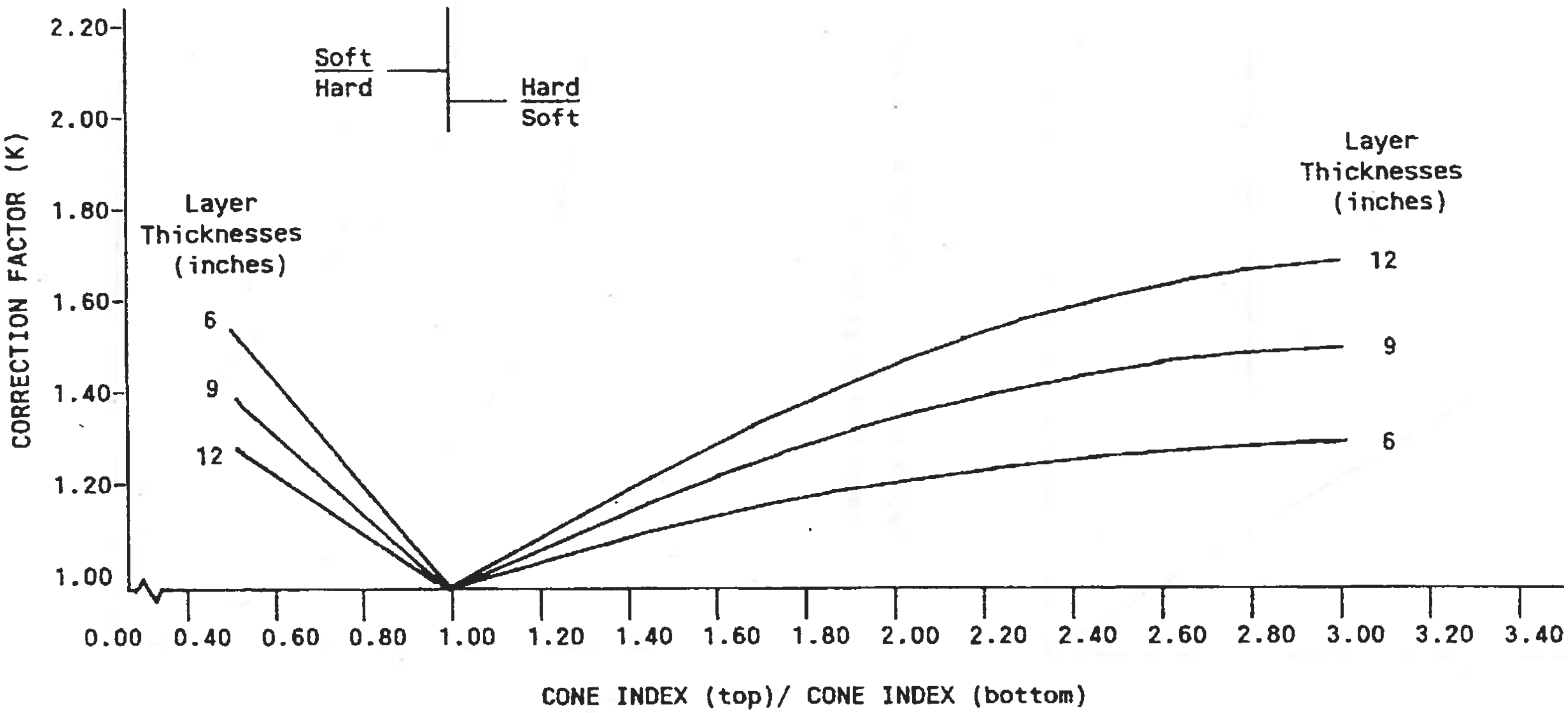


Figure 4.4 Determination of equivalent cone indices value for layered soils. (Ref. 25).

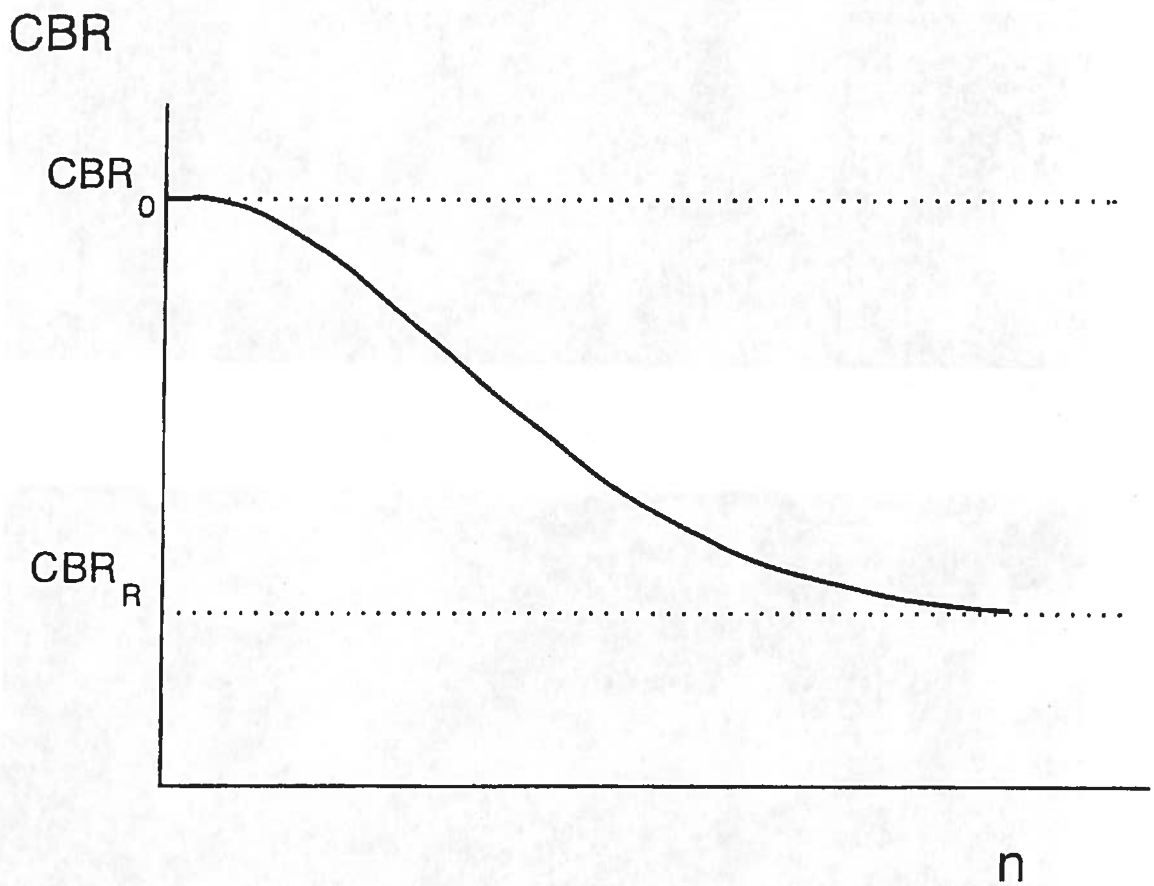
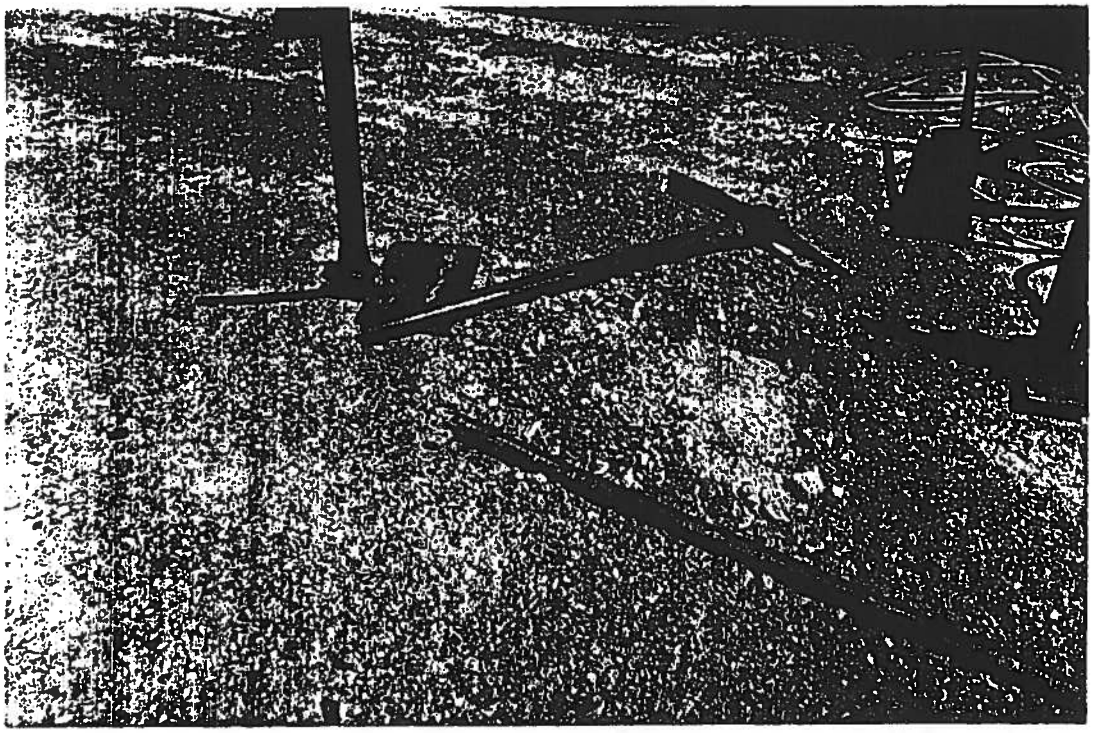
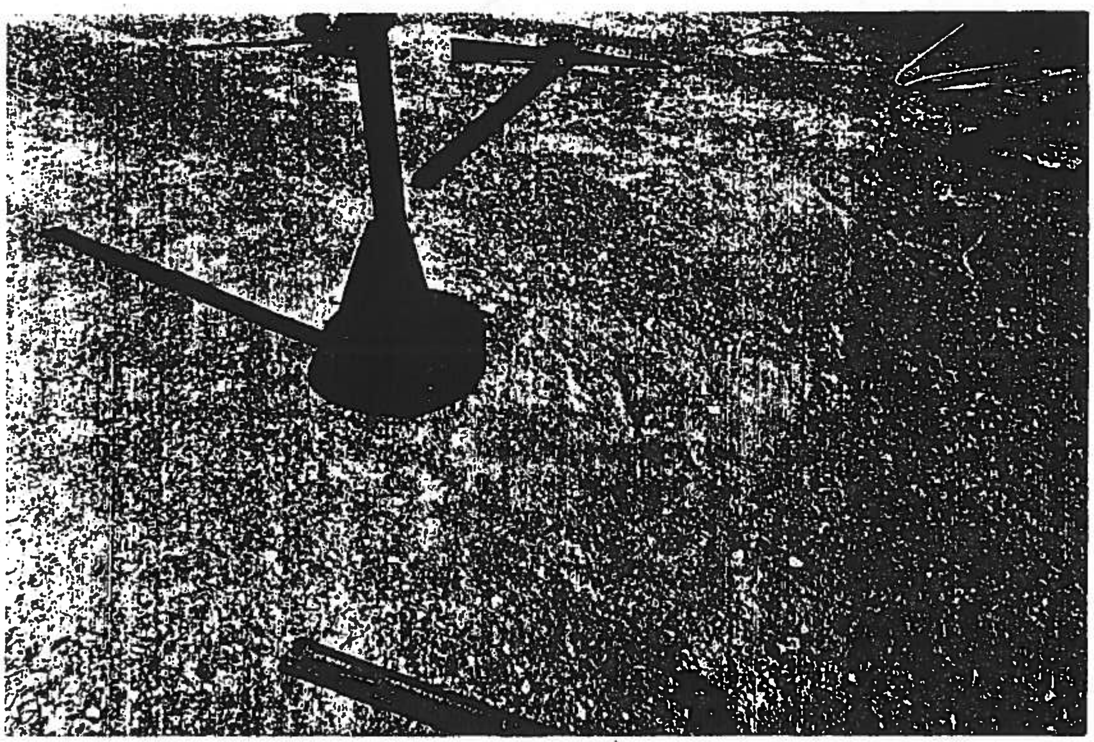


Figure 4.5 Variation of equivalent CBR value vs. number of wheel coverages.



a)



b)

Figure 4.6 Testing remolding potential of soil in site t after soil remolding:

a) Before compaction.

b) After recompaction to original soil level.

CHAPTER 5: The Research Plan and Summary

5.1 Essentials of the Research Plan

The main goal of the present research is to improve the quality of performance predictions in potentially remoldable soils under aircraft traffic. The model described in Chapter 4 is based on a number of assumptions and hypotheses stemming from the literature revue and from laboratory and in-situ observations. The behavior of the entire model closely depends on knowledge of the soil's behavior under the specific stresses created under a moving wheel load. The investigation of soil behavior will be limited in the present research to granular soils with cementation. The main reasons for this choice are:

a. The scope of the required laboratory and experimental research is wide and it is therefore necessary to limit the types of soils being investigated, in order to keep within a feasible framework of time and budget.

b. The behavior of granular soils with cementation under moving wheel loads, has hardly been investigated in the past and there is therefore a great lack of knowledge on the subject.

c. Granular soils with cementation are very common in many areas in Israel, in the Middle East, in the south of the U.S. and in many other areas in the world. This fact adds to the necessity of increasing existing knowledge in this area. The availability of soils, including landing sites (site S and site T in the south of Israel) adds to the possibility of observing the behavior of these soils under aircraft traffic, with the aim of validating the proposed model.

Despite the fact that the present research will concentrate on granular soils with cementation, it seems that the general model may also be applicable, under certain conditions, to other types of potentially remoldable soils.

Once the model is successfully developed, it may be possible to extend its application to additional potentially remoldable soils such as cohesive soils with structural strength caused by chemical bonds, and others. The two-layer model becomes less applicable as the the behavior of the soil after peak strength has been attained becomes more ductile, though for engineering purposes the model may still be applicable.

In order to prove, validate and quantify the proposed model, a research plan was developed which comprised a number of main stages, both analytical and experimental.

As the problem under consideration is very complex, it does not seem possible to provide a closed analytical solution of the problem. The analytical model will therefore be developed on the basis of numeric methods which will be iteratively applied to simulate the behavior of the soil under repeated wheel loading. At this stage, it seems that the method of finite elements is the most appropriate for application in the model as it allows flexibility in the selection of soil behavior functions and changes from one place to another and from one point in time to the next.

Along with the analytical stage, the experimental work will also be carried out, the main points of which are:

a. Laboratory tests using a shear strength testing device.
b. Laboratory moving wheel tests.
c. In situ moving wheel tests.

At the same time, an attempt will be made to adopt and test an orderly field method for determining the remolded soil strength. Such a method will mainly be studied during the moving wheel tests under field conditions.

5.2 Laboratory shear tests

At this stage, tests will be conducted with the aim of defining the constitutive laws of granular soil with cementation under a monotonous and cyclic stress regime similar to that prevailing under a moving wheel. In the context of the analytical part of the research, an attempt will be made to define a more precise stress path than is obtained through the assumption of an elastic homogeneous state, and derive a stress path capable of reflecting the stresses exerted on a point of soil in an unremolded layer of the proposed two-layer structure.

Ref. [33] presents a survey of shearing devices conducted in order to select the device which is capable of carrying out the best simulation of loading conditions under a moving wheel. The device which seems most appropriate for these needs was found to be the Hollow Cylinder Torsional Device (HCT). Ref. [34] presents a general review of the HCT device, including advantages and limitations. The most significant advantage of this instrument is its ability to control the direction of the principal stresses, a fact which is highly significant for the simulation of wheel movement over a pavement.

As the HCT device at the Technion laboratories is available, an effort will be made to carry out the laboratory shear test stage by means of this device.

The output of this stage is supposed to describe the behavior of a loaded point in the soil under various loading conditions, which information is essential for the construction of a model of soil behavior under repeated wheel loading.

The tests of this stage will be conducted on sample of granular soil taken from at least one source, which will contain natural cementation (cementation acquired through a drying process which can be emulated in the laboratory) or cementation obtained through the addition of another cementing agent (gypsum, Portland cement, etc.). The soil will be molded to achieve the dimensions of the sample required for the shear tests and will undergo a conditioning process to be determined later on.

The tests will be monotonous shear tests and cyclic shear tests under high stresses close to the soil strength (from 60% of the soil's strength and up). All tests will be carried out under relatively low confinement pressures, in order to simulate the situation pertaining under shallow wheel loading.

The main data to be measured and calculated during the tests are:

1. The behavior of the soil's stress-strain function.

2. The soil's Moduli of Elasticity.

3. The strength parameters of the unremolded and remolded soil.

4. Fatigue failure behavior under cyclic loading with loads approaching monotonous failure loads.

5. Shear strains and volumetric strains during monotonous shear and cyclic shear.

The results of this stage will be used to validate and quantify the analytical model.

5.3 Moving Wheel Tests

The moving wheel tests will be conducted during the development of the analytical model and following it, with the aim of providing feedback and validation for the model.

It is planned to carry out the moving wheel tests in two stages:

a. The laboratory stage - will be carried out by means of an existing laboratory moving wheel device. The granular material will be compacted in a mold, until the appropriate density is achieved. After a suitable conditioning process, the moving wheel device will be applied to the soil. The following parameters will be measured with progressive cycles:

   1. Accumulated rutting during a single-lane movement.
   2. Accumulated rutting during movement with lateral wander.
   3. The advance of the remolding process at the sides of the loading wheel and in a depth profile of the sample.

   The soil strength will be tested by means of a pocket penetrometer or another type of penetrometer which will allow strength tests to be carried out during the test and provide a soil strength profile. The laboratory tests will be conducted on granular material selected from one source or more which will include either a natural or an added cementational component.

   The parameters which will be varied during the test are:

   1. The moving wheel load.
   2. The quantity of the cementational material in the granular material.
   3. The density of the granular material.

b. Large scale moving wheel testing - Once conclusions have been drawn from the laboratory moving wheel stage, a series of larger scale tests will be carried out with a moving wheel. These tests will be carried out with the help of the loading carts in the possession of the Wheel-Soil Research Laboratory at the Technion, Haifa.

The tests will be carried out in one of the following locations:

1. A select field site of granular soil containing natural cementation. Such sites are mainly widespread in the south of Israel. The selection of such a site involves a number of drawbacks stemming from the limitations of distance, time required for execution and the expected variance due to the natural character of the site. However, such field work may provide clearer answers as to the actual behavior of the soil.

2. Use of the agricultural channel - the soil will be worked by drying or adding a cementing factor only, until the required qualities are obtained. The advantages of this site lie in its ability to provide clear results and the lower variance of the observations because of the control of the working process.

The parameters which will be varied during the tests are:

1. The tire pressure of the loading wheel.
2. The total wheel load.
3. Movement in one lane and movement with lateral wander.

The measurements which will be conducted throughout the testing process will be:

1. The stresses exerted on the soil by the loading wheel.
2. The lateral soil profile with the progress of the test.

3. The strength of the soil and the change of the strength with depth in remolded and unremolded areas.

At this stage of the large scale moving wheel tests, various attempts will be made to establish and test a field method for determining the in-situ strength of the remolded soil, in accordance with the guidelines set forth in section 4.3

The tests will be carried out at a constant free rolling speed of up to about 3m./sec. If possible, the tests will be expanded so as to examine the soil's reaction to braking wheel movement.

As the aim of the moving wheel tests, both at the laboratory level and at the level of the loading carts, is to validate and receive feedback relevant to the analytical or numeric model, the tests will be carried out on a limited scale as dictated by budgetary constraints.

5.4 Summary

The prediction methods for the performance of unsurfaced runways under aircraft traffic, often yield erroneous estimations of the predicted number of cycles until failure of the runway. One of the common causes for these incorrect estimates are the strength change processes which take place in certain types of soil during repeated aircraft loading.

Field methods which have been developed in order to evaluate the soil strength change potential are simplistic and only applicable to soils with a high clayey component. These methods provide no more than a rough estimate of the soil's strength after its remolding, and do not address the issue of gradual soil strength change during movement. A review of the literature did not elicit any model capable of describing the process of soil strength change, in a depth profile, during repeated wheel movement. The main aim of the present research work is to develop such a model capable of providing better and more reliable practical solutions for predicting the performance of unsurfaced aircraft runways.

The performance of the soil profile under wheel movement totally depends on the behavior of the specific soil under the pertaining stress and strain conditions. The loading of the soil by the wheel creates special loading conditions in the layer immediately under the surface, including special stress paths and low confinement pressures. Thus, the developing of the model requires a preliminary stage of defining the behavior of the soil element under given loading conditions. In this area, too, the review of the literature did no reveal a model relevant to the behavior of potentially remoldable soils under loading conditions similar to those detailed. In order to keep to a clearly defined research framework, it was therefore decided that the research emphasis would be on a certain class of soils which are granular soils with cementation. The fact that such soils are widespread in many areas in the world, only adds to the need to study them.

The basic assumptions which can be derived from the literature survey are:

a. The behavior of granular soils with cementation under loading within low confinement pressures, is brittle. Once the soil has attained its peak strength there is a sharp decrease in strength toward the residual value, and the cementation is broken.

b. The added strength in granular soils with cementation is mainly cohesive. Soil shear is accompanied by the annulment of the cohesive factor (C in the Mohr-Coloumb equations), while the internal angle of friction remains almost constant.

c. It is also possible to construct a fatigue function for lower stresses than the peak strength, in order to describe the relationship between the stress level and the number of cycles required in order to remold the added soil strength.

On the basis of the literature review and the observations made both in the field and in the laboratory, general guidelines have been constructed for the proposed model and the description of the strength profile of a potentially remoldable soil under cyclic wheel loading. The cyclic wheel loading remolds the part of the soil immediately under the surface, thus giving rise to a two-layer structure wherein the bottom layer retains the original soil characteristics, while the upper layer has been remolded and its cementational element gone. The thickness of the remolded layer increases gradually until it stabilizes in accordance with the soil's strength parameters and the nature of the loading.

This general picture of the model is only hypothetical and based on a number of assumptions which require validation, mainly in regard to the basic behavior of granular soil with cementation under load in the given conditions. Accordingly, the research program constructed, includes the following main stages:

a. The formulation of an analytical model to quantitatively describe the strength behavior of the soil profile beneath the loading wheel. The model will make use of analytical and numerical methods and will address the effect of repeated passes on the progression of the process.
b. Laboratory testing of the behavior of granular materials with cementation under monotonous and cyclic loading, and under stress paths and confinement pressures as close as possible to those prevailing under a moving wheel. The results of this stage will serve for essential validation of the proposed two-layer model and for calibrating the quantitative model.

c. Laboratory-scale moving wheel loading tests, and moving wheel tests approximating reality, will serve to provide feedback and to validate the results obtained in the two previous stages. The work with the laboratory moving wheel will be carried out when required, throughout the research period. The moving wheel tests in the agricultural channel or in the field, will be concentrated toward the end of the research in order to validate and approve the obtained model.

The main product of the research should be a quantitative model which will describe the behavior of the strength profile of granular soil with cementation under cyclic wheel loading. Once the development and validation of the model have been completed, and using the results obtained from the moving wheel tests, an attempt will be made to define a prediction method which makes basic use of existing design methods, while at the same time addressing the process of strength change during repeated loadings. This issue may be integrated within a test method to be constructed and tried in the course of the research, for the purpose of determining the in-situ strength of the soil after remolding. In future, it may be possible to re-apply stage A (the laboratory study) of the research to other classes of potentially remoldable soils. The results of such a laboratory study will establish the applicability of the model to the description of the behavior of other soils under moving wheel loading.

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